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WEAK ROCK MASS MODEL FOR SUPPORT OF SURFACE CROWN PILLAR AT LES MINES SELBAIE

DSS Project No. 15 SQ. 23440-5-9017

Phase II - Model Development and Model Calibration

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WEAK ROCK MASS MODEL FOR SUPPORT OF SURFACE CROWN PILLAR AT LES MINES SELBAIE

DSS Project No. 15 SQ. 23440-5-9017

Phase II - Model Development and Model Calibration

1.0 INTRODUCTION

A satisfactory design approach to ground control in surface crown pillar mining is a pressing yet unresolved subject. Various mines throughout Canada, but particularly in Quebec, now operate with some form of surface crown pillar configuration. Considerable interest has been generated in developing a design approach which provides for optimum recovery of pillar ore reserves while assuring adequate safety. Much research activity has already been generated, with CANMET playing a leading and significant role in spear-heading directions for rock mass evaluation and computer modelling

In April 1985, C. Mirza Engineering Incorporated was retained by Supply and Services Canada, on behalf of CANMET, to conduct a three-phase study of the surface crown pillar at Les Mines Selbaie in the Province of Quebec.

The first phase, completed in August 1986, consisted of a field and laboratory investigation of a near-surface crown pillar at Les Mines Selbaie.

Phase I acted as a prelude to the second phase which involved the development and calibration of a weak rock mass model for predicting stability and support requirements.

This report presents the results of the weak rock mass model devlopment and the calibration study in compliance with the requirements for completion of Phase II of the study.

2.0 STUDY OBJECTIVE

Les Mines Selbaie is a 1650 tpd copper mine located in Brouillon Township, northwestern Quebec, approximately 60 km northwest of Joutel and 40 km east of the Ontario/Quebec border (see Fig. 2-1).

The rock mass in the ore body is severely fractured and is generally covered by less than 45 metres of overburden. In Quebec, the crown pillar is generally considered to be three times the width of the orebody and any mining of it requires approval from the Provincial Mining Authorities.

At Les Mines Selbaie, the richest ore deposits are located within the upper weathered, fractured and kaolinized zone lying immediately below the overburden. Therefore, special dispensation has been received from the Provincial Mining Authorities to leave a minimum cover of 7.5 m.

The overall Study has been divided into three distinct Phases as follows:

Phase I) Drilling, sampling and laboratory testing,
Phase II) Modelling,
Phase III) Ground monitoring during a field trial of the proposed mining method

During Phase I of this study, three 115 mm diameter boreholes were drilled at the site (Fig. 2-2). The extent as well as the index and mechanical properties of the overburden till and the kaolinized zone in the orebody zone were established. Two

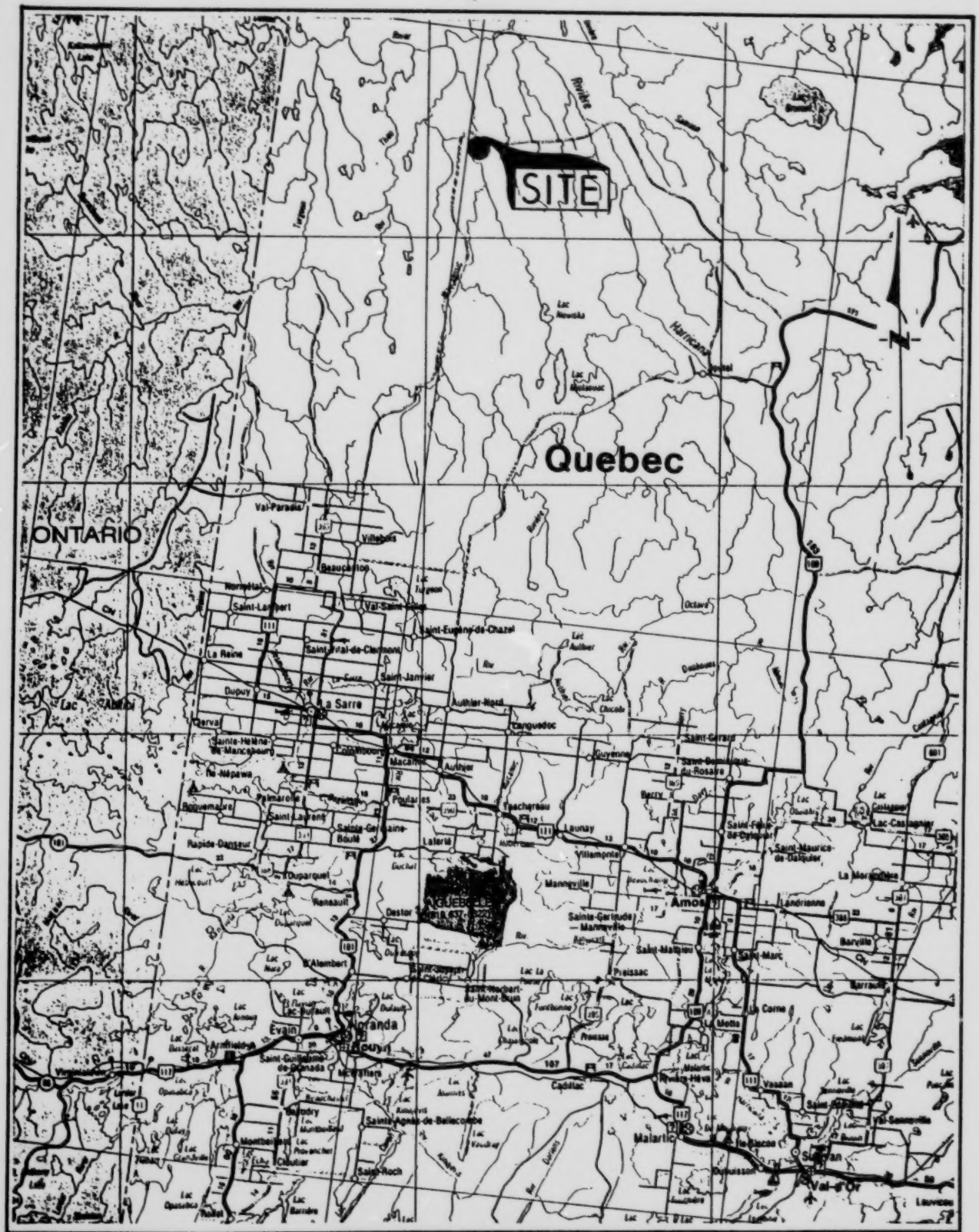


Figure 2-1 Key Plan

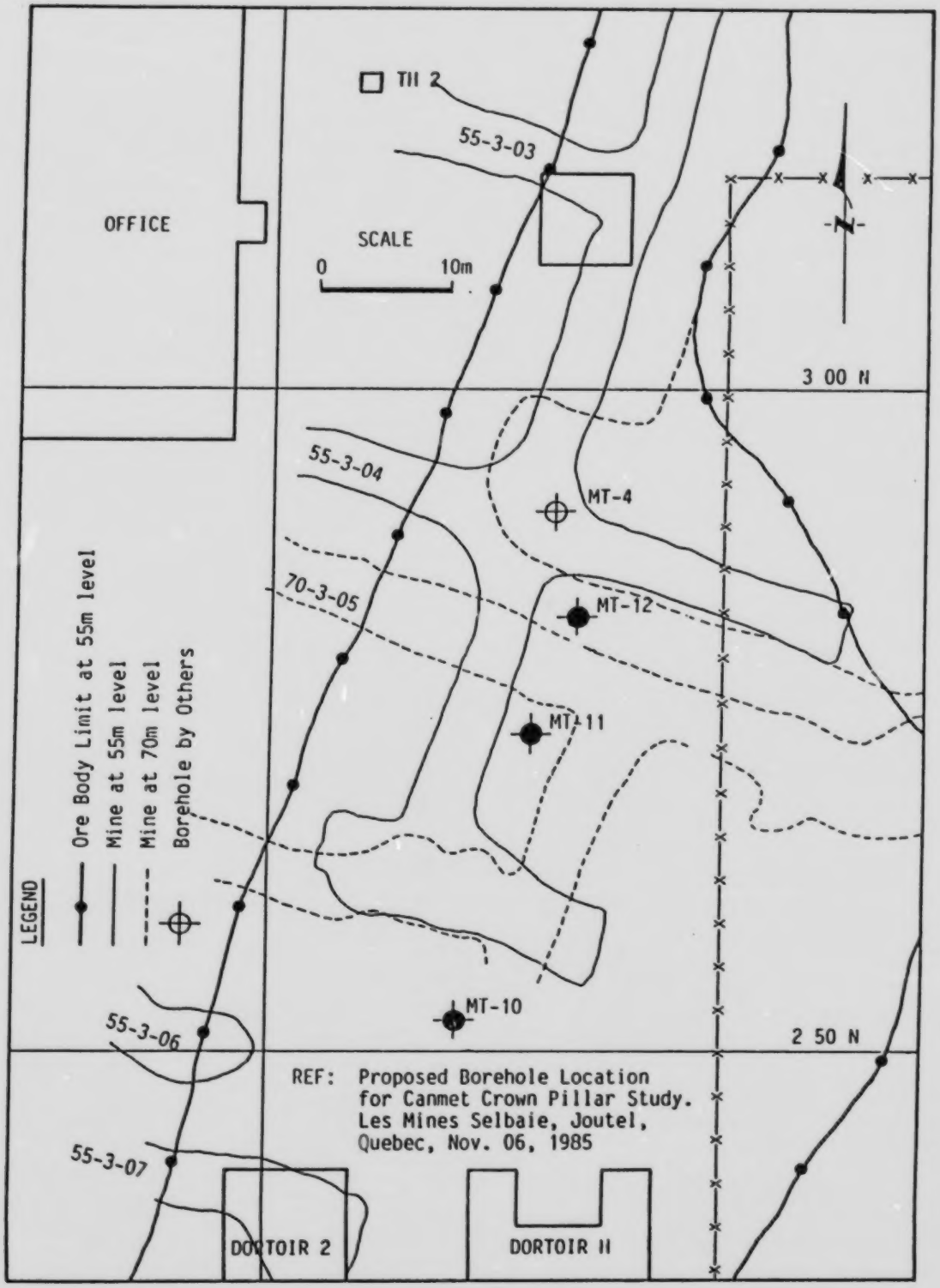


Figure 2-2 Plan Showing Borehole Locations

piezometers were installed in one of the boreholes to determine the groundwater conditions at the site.

With the completion of the field and laboratory study (Phase I), this study, Phase II, was authorized by CANMET to develop a weak rock mass model for Les Mines Selbaie for the purpose of determining the stability and support requirements of the surface crown pillars at this Mine.

Phase II has the following objectives:

a. develop/select in collaboration with CANMET, a weak rock mass model capable of predicting safe mine extraction.

b. calibrate the model with data from a previous cave-in at the mine.

c. develop stability and support requirements of the surface crown pillars.

3.0 SITE DETAILS

3.1 Location

Les Mines Selbaie is located in Brouillon Township, northwestern Quebec, approximately 60 km northwest of Joutel and 40 km east of the Ontario/Quebec border (Fig. 2-1).

3.2 Geology

The mine is located within the Chibougamou-Metagami section of the Abitibi volcanic belt. The geology of the area has been well documented by Sinclair and Gasparrini, 1979 and Deptuck et. al., 1982.

Generally, the surrounding area is covered by overburden consisting of sands, gravels, till materials and organics. The soil (overburden) stratigraphy is presented in Fig. 3-1. A clayey glacial till known locally as the Cochrane Till (often characterized by NW-SE low-relief fluting of the ground surface) is found here. The bedrock generally consists of a series of tuffs and breccias, bedded chert-pyrite and volcanoclastic debris located near the southwestern margin of a plutonic complex known locally as the Brouillon Granite.

The area of primary investigation (boreholes MT4 - MT10) is covered by sand and gravel fill with virtually no vegetative cover. It is generally flat and dips slightly to the south.

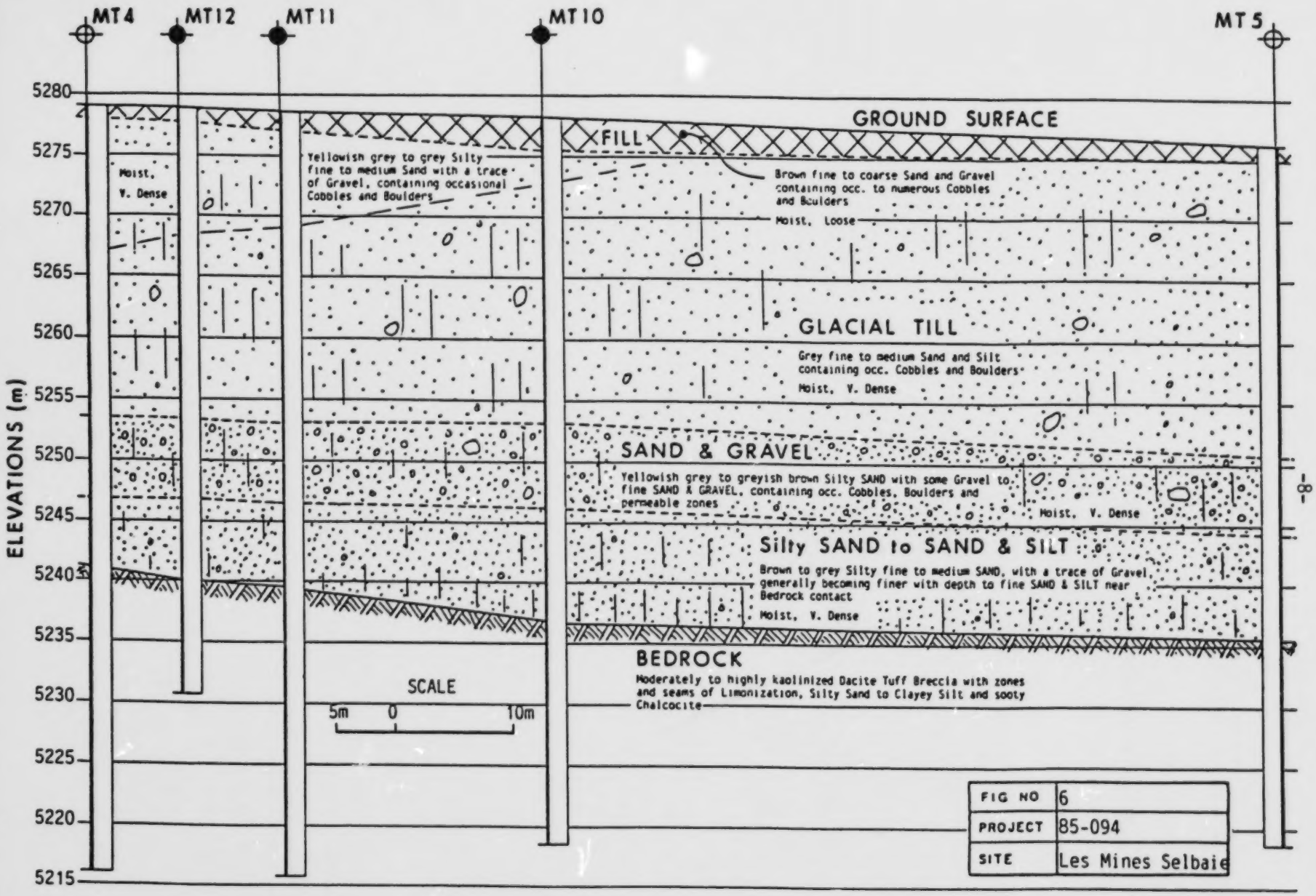

MT4
MT12
MT11
MT10
MT5
5280
5275
5270
5265
5260
5255
5250
5245
5240
5235
5230
5225
5220
5215
ELEVATIONS (m)
GROUND SURFACE
FILL
Moist, V. Dense
Yellowish grey to grey Silty fine to medium Sand with a trace of Gravel, containing occasional Cobbles and Boulders
Brown fine to coarse Sand and Gravel containing occ. to numerous Cobbles and Boulders
Moist, Loose
GLACIAL TILL
Grey fine to medium Sand and Silt containing occ. Cobbles and Boulders
Moist, V. Dense
SAND & GRAVEL
Yellowish grey to greyish brown Silty SAND with some Gravel to fine SAND & GRAVEL, containing occ. Cobbles, Boulders and permeable zones
Moist, V. Dense
Silty SAND to SAND & SILT
Brown to grey Silty fine to medium SAND, with a trace of Gravel, generally becoming finer with depth to fine SAND & SILT near Bedrock contact
Moist, V. Dense
BEDROCK
Moderately to highly kaolinized Dacite Tuff Breccia with zones and seams of Limonization, Silty Sand to Clayey Silt and sooty Chalcocite
SCALE
5m 0 10m
FIG NO 6
PROJECT 85-094
SITE Les Mines Selbaie

Figure 3-1 The Overburden Stratigraphy

Approximatly 40 m of overburden overlies the bedrock. The uppermost level of the mine is located at a depth of 55 m. The mine is situated in a tabular orebody dipping at 50 degrees and associated with a major northeasterly-striking fault. The fault consists of quartz-sulphide vein systems up to 20 cm in width. The veins occur within several elongate lenses of heavily-fractured rock, up to 30 m wide which lie in the hanging wall of the fault.

Supergene alteration (Sinclair and Gasparrini, 1979) has played an important role in the enrichment of ore values within the deposit and has been found up to the maximum depth yet explored (200 m). The fault system has provided an excellent channelway for descending meteoric water and, as a result, such alteration is widespread. Supergene alteration is typically characterized by the presence of sooty chalcocite, chalcopyrite almost completely replaced by bornite, limonite and relatively high amounts of native copper, all of which occur in a porous, sugary quartz and kaolin gangue.

3.3 Material Properties

The material properties of the crown pillar at Les Mines Selbaie were determined by both field and laboratory testing. The shear strength characteristics of rock joints at Les Mines Selbaie were determined by CANMET (Swan et. al., 1984) through a thorough laboratory testing program. Subsequently, index tests, compressive strength tests and direct shear tests were conducted on rock core samples obtained from the three boreholes drilled during Phase I of this project. Borehole pressuremeter tests and in-situ stress measurements were conducted by others, to determine the field stresses and deformation modulii of the orebody (Arjang, 1984; Swan, 1985; Bétournay, 1987).

In general, the orebody is composed of three types of materials. They are (1) a rock-type material, (2) a soil-like material and (3) infilling material. The distinction between the rock-like material and the soil-like material is based on a classification system developed during Phase I of this project. The classification system ranks the orebody material from 1 (very hard) to 11 (very soft) on the basis of its approximate compressive strength. A description of this classification system is presented in Table 3-1. The volume ratio between the soil-like material (rank of 7 to 11) and the total volume of the soil-like and rock-like (rank of 1 to 6) materials is defined in this project as the volumetric ratio of the orebody. The results of three scanline surveys (locations shown on Fig. 3-2) located in the orebody zone at the 55 metre level indicate that the volumetric ratio around future crown pillars will be about 30 per cent. This volumetric ratio is relatively high compared to the 15 per cent volumetric ratio reported earlier (Swan, 1985).

RANK	CHARACTERIZATION	FIELD METHOD OF DETERMINING RELATIVE STRENGTH	AVERAGE VOLUME (%)
1	Very Strong	> 1 hammer blow to break	9.7
2	Strong	1 hammer blow to break	17.4
3	Moderately Strong	5 mm indentations with pick	12.4
4	Moderately Weak	Cannot be cut by hand	6.3
5	Weak	Crumbles under firm blows with pick	14.1
6	Very Weak	Broken in hand with difficulty	9.4
7	Very Stiff	Indented by fingernail	6.6
8	Stiff	Cannot be moulded in fingers	5.7
9	Firm	Difficult to mould in fingers	7.2
10	Soft	Easily moulded in fingers	10.4
11	Very Soft	Can be squeezed between fingers	0.8

Table 3-1 Ranking Scheme and Average Volume of each Rock Type

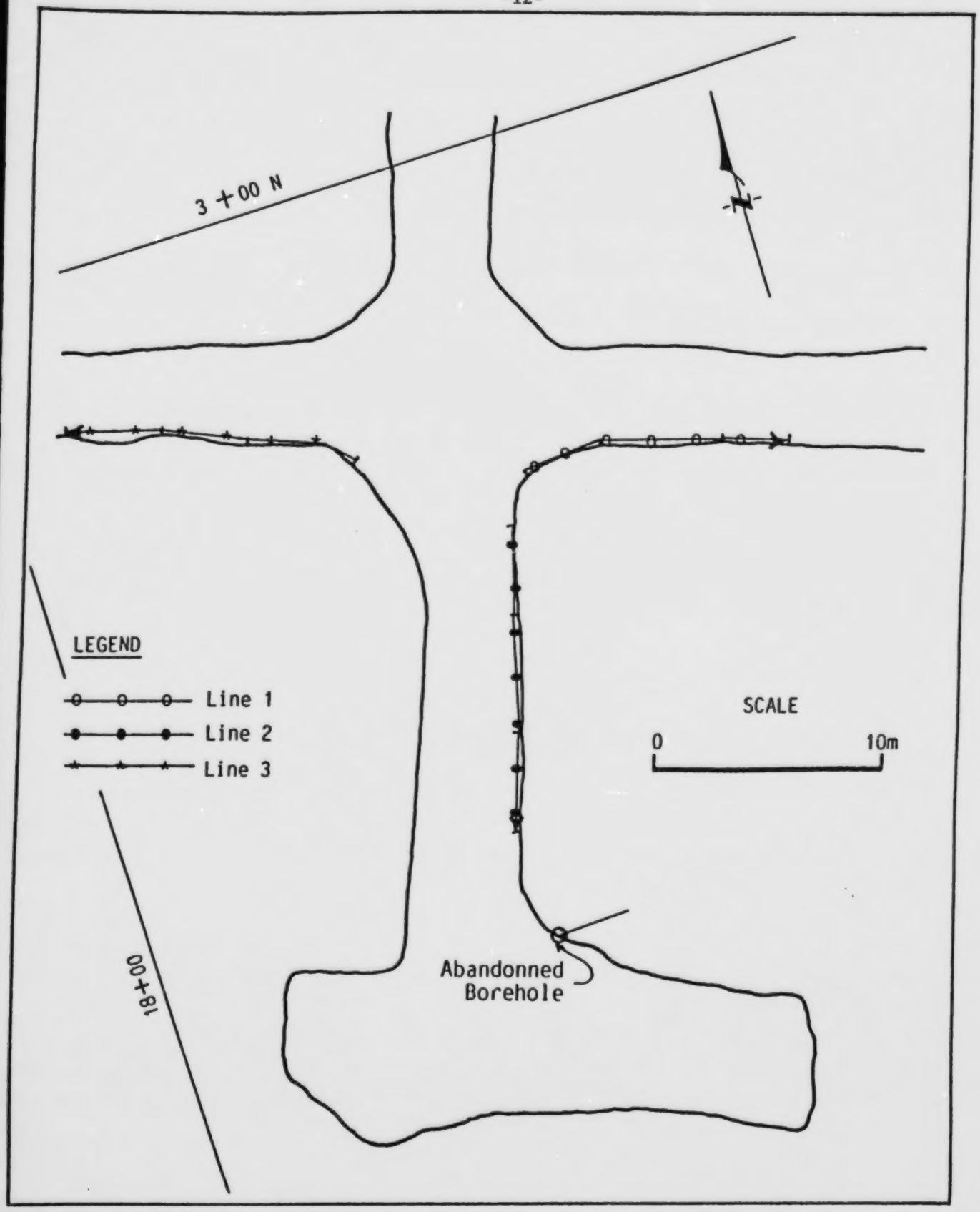


Figure 3-2 Location of Line Surveys and Abandoned Borehole

3.3.1 Laboratory Test Results

3.3.1.1 Rock-like Material

The rock-like materials comprise about 70 per cent of the orebody by volume and consist of a range of materials from fresh dacite tuff to highly decomposed, kaolinized (or limonized) dacite tuff breccia (or siliceous dacite tuff breccia) containing lenses or bands of moderately decomposed quartz and faintly decomposed chalcocite.

Unit weights were found to vary from 24.2 to 29.2 kN/m3 and averaged about 25.6 kN/m3 for all types of rock-like materials.

The slake index, I2, of the weaker rock-like material (rank of 6 to 7), after 2 cycles ranged from 68.9 to 98.3 per cent (average of 87.9 per cent). The unconfined compressive strength conducted on various grades of material with point load, Schmidt hammer and unconfined compressive strength tests is plotted in Fig. 3-3.

The modulii of deformation of the rock-like materials were determined using correlations with the Schmidt rebound number and from the stress-strain curves from the unconfined compression tests. The results are plotted in semi-log form in Fig. 3-4 against the rank of the material.

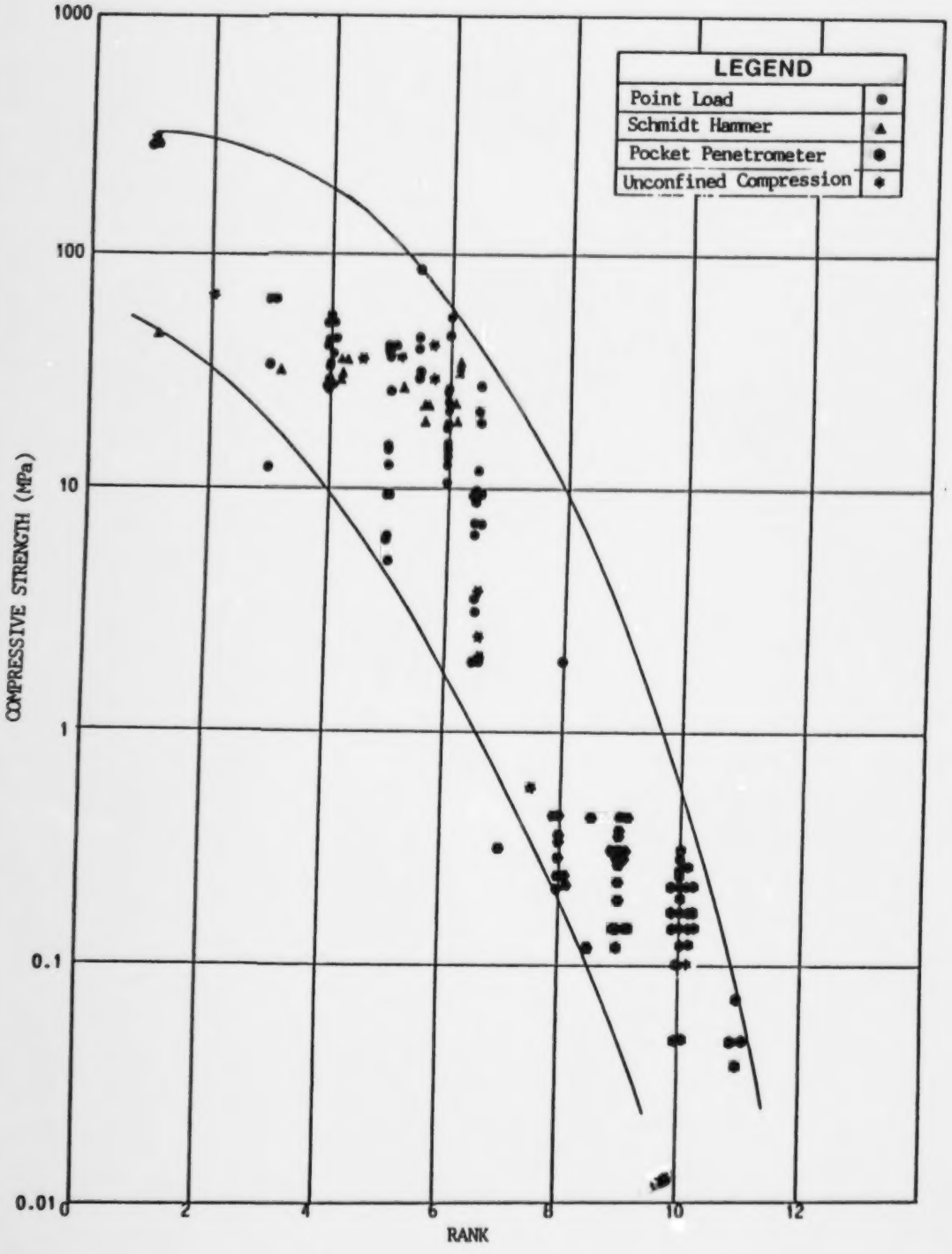


Figure 3-3 Unconfined Compressive Strength versus Rank

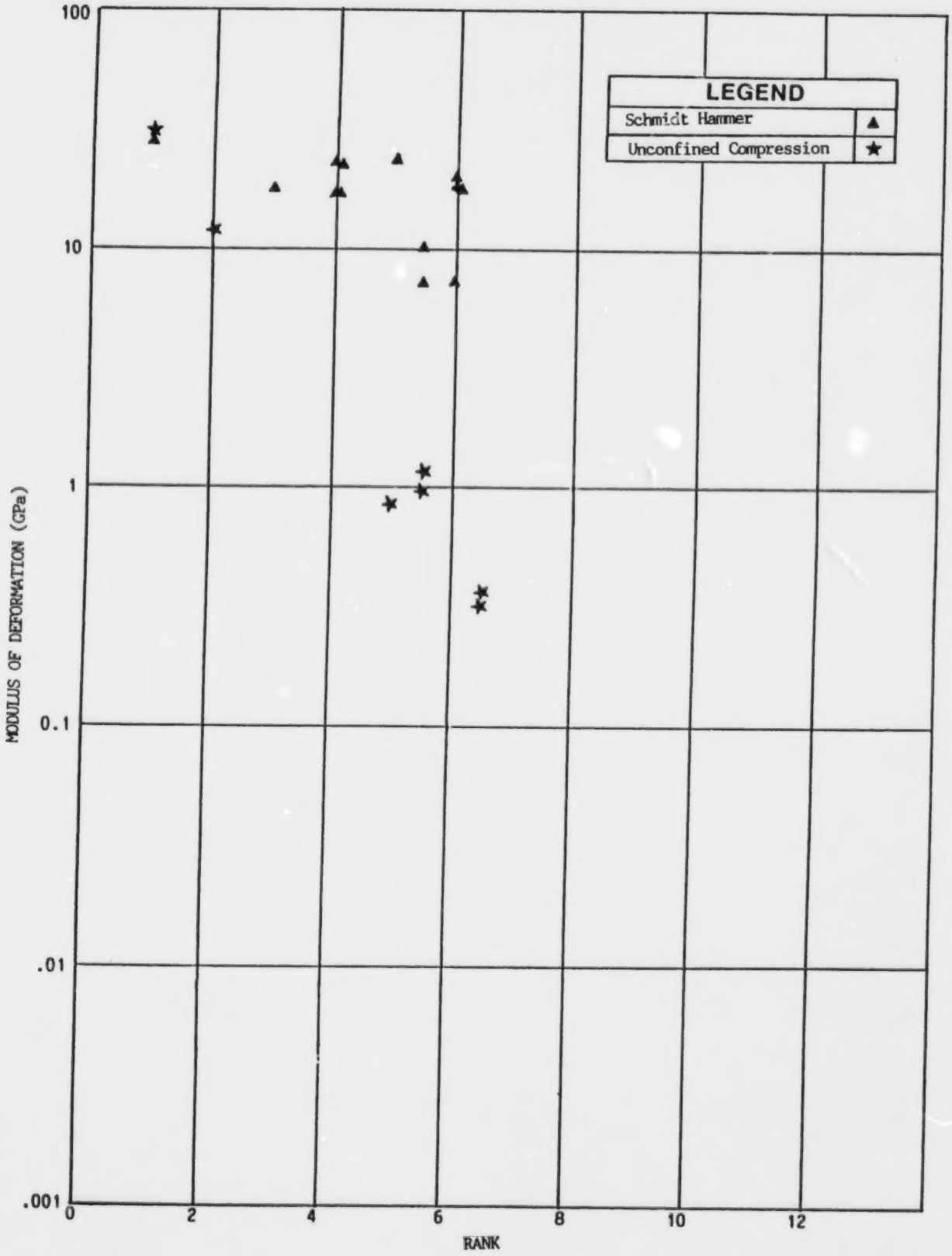


Figure 3-4 Modulus of Deformation versus Rank

3.3.1.2 Soil-Like Material

The soil-like "rock" comprises about 30 per cent of the orebody. The material consists of highly to completely decomposed, kaolinized (or limonized) dacite tuff breccia (or siliceous tuff breccia). It can generally be characterized as gravel-sized angular pieces of rock in a silty clay to sandy silt matrix. The soil-like rock has also been used to refer to zones of decomposed sooty chalcocite stringers.

The moisture content generally averaged about 12 per cent over the orebody. The moisture content of the white kaolinite " rock" encountered in the borehole cores and underground varied from 15 to 24 per cent and averaged about 18 per cent.

Atterberg limit tests were carried out on samples of the white kaolinite "rock". The material is essentially a silt to silty clay of low plasticity (CL-ML).

The activity of this material ranged between 0.54 to 1.97 which is higher than might be expected for a pure kaolinite clay (activity of kaolinite clay = 0.38; Lambe and Whitman, 1969).

Unit weights of the soil-like materials ranged from 18.6 to 25.9 kN/m3 (average of 23.9 kN/m3).

The unconfined compressive strengths of the soil-like core samples are plotted in Fig. 3-3 along with similar information obtained from the line surveys underground.

Finally, a special observational-type test was carried out on a sample of soil-like "rock" to monitor its behavior when subjected to soaking. A core sample of dry material was weighed and then placed in a beaker filled with water. Almost immediately, the sample began to slake, bubbles were given out and a sand and silt layer began to form at the bottom of the jar. The slaking decreased substantially after about 5 minutes, at which time the intact piece of "rock" was placed into a drying oven. Subsequent weight measurements indicated that its weight loss was about 60 per cent .

3.3.1.3 Infilling Materials

Silt and Sand

There is a great similarity between the grain size distributions which were obtained for the silt and sand infilling and the silty sand to sand and silt layer immediately above the bedrock contact. This indicates that these sand infillings are probably the result of overburden materials being carried down through joints and fractures in the bedrock by descending meteoric water.

Moisture content tests carried out on this material showed its moisture to be 12 to 15 per cent , averaging about 14 per cent .

Clay and Silt

A relatively thick (40 mm - 1.5 m), soft to very soft, bluish-grey clay and silt infilling associated with faulting was also found.

Hydrometer tests show this material to have an average of about 47 per cent silt and 25 per cent clay by weight.

The moisture content varied from 17.7 to 23.4 per cent (average of 20.6 per cent). Atterberg Limits test results indicate that the material behaves as a clay of low to intermediate plasticity (CL-CI). Activities varied from 0.28 to 1.9. The low plasticity of the clay indicates it is probably not smectitic.

Pocket penetrometer tests indicated compressive strengths ranging from 48 to 72 kPa.

3.3.1.4. Rock Joints

Rock joint shear properties at Les Mines Selbaie were estimated by CANMET (Swan, 1984) from predictive models (Komornik et. al., 1970). Ten block samples each featuring a selected joint surface with a minimum dimension of 0.2 x 0.5 m were obtained from the mine. The model described in Komornik et. al. was used for the estimation of the shear properties of the rock joints. The results are presented in Table 3-2.

The hydraulic conductivity of the overall orebody including the soil and rock like materials, is determined to be 5 x 10-4 cm/s (Bryce, 1985).

Joint Number	ϕ_r (Degs.)	U_u	$1/C_{22}$ (MPa/m) σ_1 = 1 MPa	100 kPa
1	4.7	0.33	100.5	8.95
2	5.0	0.32	93.2	8.30
3	6.2	0.33	200.3	17.85
4	4.9	0.36	233.5	20.81
5	6.8	0.26	216.1	19.26
6	5.9	0.34	230.4	20.54
7	5.9	0.33	171.2	15.26
8	7.5	0.28	211.6	18.86
9	4.2	0.44	162.0	14.44
10	9.7	0.37	614.6	54.78

NOTE: ϕ_r = residual friction angle
i = surface slope angle
μ_u = tan (ϕ_r + i)
C_{zz} = shear compliance

Table 3-2 Selbaie Joint Shear properties (Swan et.al. 1984)

3.3.2 Field Testing

The deformation properties of the orebody at the 55 m level were determined by pressuremeter tests conducted by CANMET (1985). The pressuremeter used had dimensions of 1 m x 115 mm in diameter (Swan and Vaillancourt; 1983, Herget, 1981). The location of the test hole is shown in Fig. 3-5. Eight tests were conducted along the borehole. Assuming a Poisson ratio of 0.4, the deformation modulus values calculated for the test results are given in Table 3-3. In 1987, a dilatometer test was conducted by Bétournay (1987) in Borehole MT11 at 55m level of the orebody. With a Poisson ratio of 0.4, the deformation modulus was calculated to be 0.14 GPa. For the silty sand overburden at the 37.5 m level in Borehole MT12, a deformation modulus of 0.039 GPa (ν=0.4) was obtained by a pressuremeter test.

The large range and variation with depth of the deformation modulus values probably indicates the absence of a consistent pattern of the occurrence of the kaolinized material.

Field stress determinations were conducted by CANMET (Arjang, 1984) with an overcoring technique using CSIRO hollow inclusion triaxial strain cells. Tests were done at the 70, 180 and 290 m levels.

Because of various problems encountered during the test, three sets of test results were discarded. A list of the stress value determinations is shown in Table 3-4. In general, the horizontal stress is higher than the vertical stress.

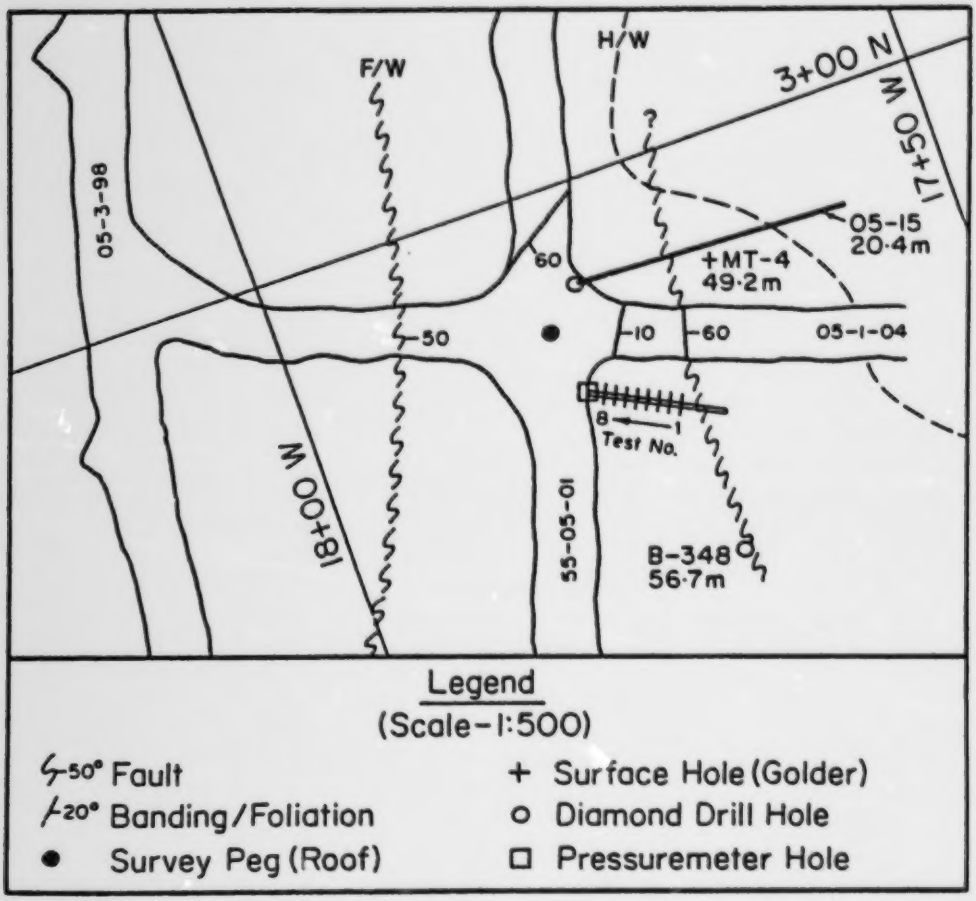


Figure 3-5 Locations of Pressuremeter Tests (Swan, 1985)

TEST	COLLAR DISTANCE (m)	V_o (cm^3)	2r (mm)	DEFORMATION MODULUS (GPa)
SELH01	6.42	149.5	122.3	1.25
SELH02	5.66	144.6	122.2	0.38
SELH03	4.92	159.9	122.5	0.67
SELH04	4.16	134.6	122.1	0.49
SELH05	3.42	287.1	124.4	0.28
SELH06	2.66	154.1	122.4	0.35
SELH07	1.92	147.3	122.3	2.46
SELH08	1.16	143.9	122.2	1.46

Table 3-3 Pressuremeter Results Summary (Swan, 1985)

Depth (m)	Test No.	Setting depth from collar (m)	Stress Components (MPa) (Standard errors in brackets)						Stress Ratios ($\sigma H/\sigma V$)		
			Horizontal σ E-W	Horizontal σ N-S	Vert. σ	Shear Stresses τ N-S/E-W	Shear Stresses τ E-W/V	Shear Stresses τ V/N-S	K_{N-S}*	K_{E-W}	$K_{Ave.}$
70	: SB3-1	5.92	0.80 (.04)	1.97 (.10)	1.55 (.04)	-1.67 (.05)	0.63 (.03)	-0.30 (.04)	1.27	0.51	0.89
180	: SB4-1	5.98	6.57 (.13)	3.43 (.05)	1.97 (.06)	-0.66 (.05)	-0.86 (.08)	0.19 (.03)	1.76	3.38	2.57
	SB4-2	7.14	11.33 (3.08)	4.66 (.81)	2.09 (.85)	-1.56 (1.04)	-0.74 (1.03)	0.04 (.43)	2.22	5.42	3.82
	SB4-3	7.84	11.08 (.27)	7.21 (.15)	6.57 (.11)	-1.58 (.13)	0.23 (.13)	-0.20 (.06)	1.06	1.64	1.35
290	: SB1-1	1.81	13.31 (.11)	7.45 (.05)	6.40 (.05)	-2.62 (.05)	-1.88 (.05)	0.00 (.03)	1.66	2.07	1.61
	SB2-1	4.55	5.87 (.07)	3.55 (.03)	4.60 (.04)	-0.26 (.03)	1.68 (.05)	0.60 (.02)	0.72	1.27	0.99

* $K_{N-S} = \frac{\sigma N-S}{\sigma V}$; $K_{E-W} = \frac{\sigma E-W}{\sigma V}$; $K_{Average} = \frac{K_{N-S} + K_{E-W}}{2}$

Table 3-4 Stress Components and Stress Ratios from Triaxial Strain Cell measurements on the 70, 180 and 290m levels, Selbaie Mine (Arjang, 1984)

3.4 Groundwater Conditions

During the initial exploration, the groundwater table was located at a depth of 3 m from the ground surface (Bryce, 1985). The presence of a high groundwater level would have a significant effect on the safety of the mine and the economy of stoping. The rate of groundwater flow into the mine was determined to be 90 to 130 l/sec.

Although the groundwater problem in most mines in the Canadian Shield is controlled by means of grouting, dewatering by drainage was implemented in Les Mines Selbaie (Bryce et. al., 1983).

Drainage drifts were excavated along the hanging wall of the main vein at the 120 and 180 metre levels (Fig. 3-6). Drainage holes were then drilled on a 40 by 40 metre pattern. A total of 11,700 metres of drainage holes were drilled during this program.

Drainage ditches were provided on the surface to carry away the surface run-off water. This drainage is aided by the surface contours which slope away from the mine in three directions and by a very impermeable clay-like till layer located immediately under the muskeg. The clay-like layer limits the amount of seepage into the overburden and the orebody.

The groundwater level was lowered to 180 metres below the ground surface three years after the implementation of the dewatering program in 1978 (Fig. 3-7).

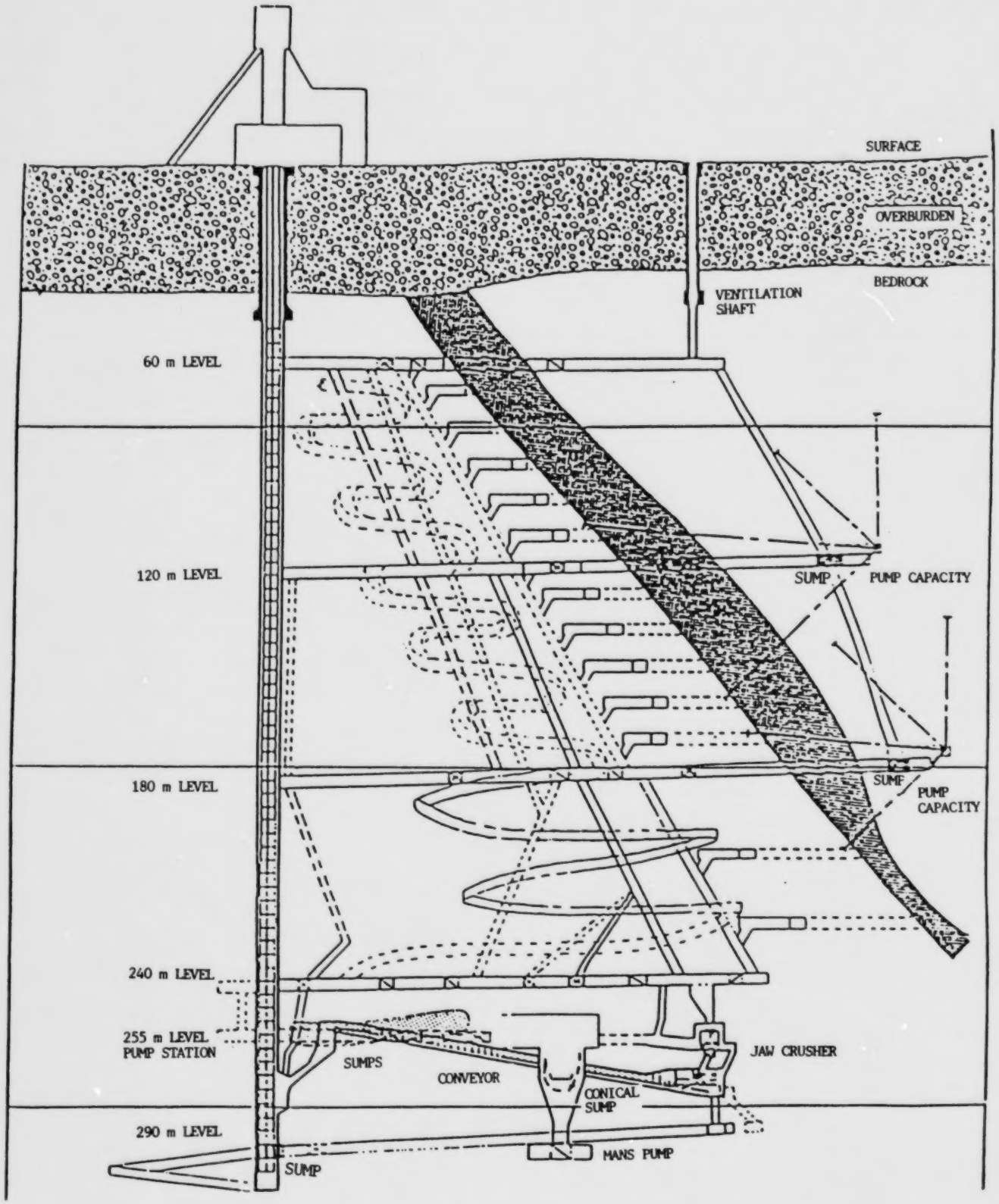


Figure 3-6 Mine Section Showing Location of Drainage Drifts (Bryce, 1985)

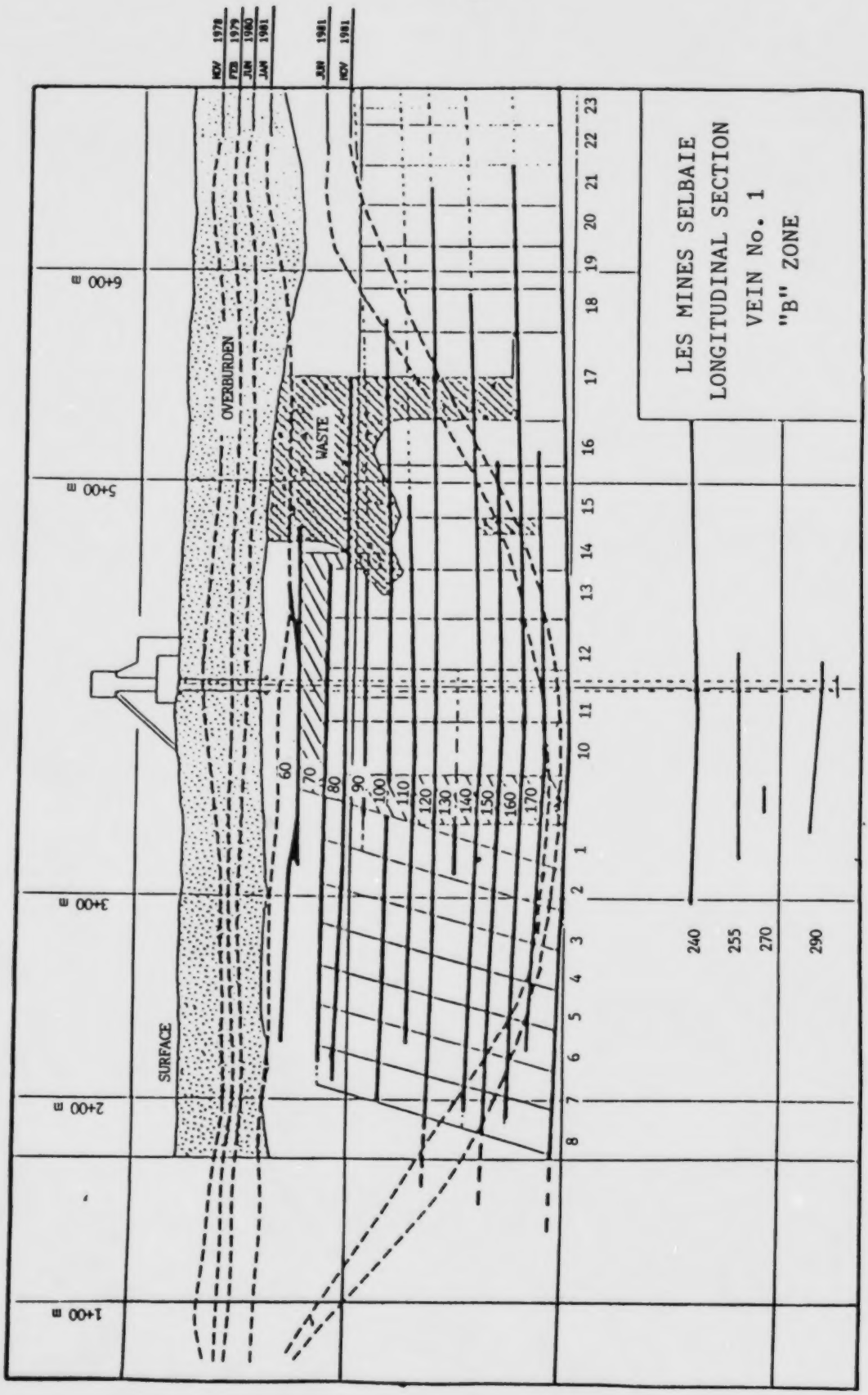


Figure 3-7 Water Table Drawdown (Bryce, 1985)

3.5 Mining Methods

During preliminary exploration operations, five possible mining methods were considered. They were:

1. Sub-level retreat
2. Caving
3. Cut and fill
4. Diamond-shaped blasthole
5. Flat back blasthole

The sub-level retreat method was rejected because the dip of the orebody was not steep enough. With the caving method, the possible induced ground surface subsidence may effect the permanent installations on the ground surface which are located within 30 metres (horizontally) of the ground pillar subcrops. The cut and fill method was attempted but was abandoned because ground conditions deteriorated during initial mining. The stope backs could not be supported over the full width of the stope. However, this method still remains a possible alternative for extraction in good quality ground.

For the diamond-shaped blasthole method, owing to the cohesiveness of the kaolinized ore, the material can sustain fill angles of more than 70 degrees. Thus, significant amounts of ore could not be recovered from the hanging wall side of the orebody unless some of the backfill from the previous stope had been undercut before this extra muck could be removed. This extra work is expensive and exposed miners to dangerous conditions. The extra ore was estimated to be about 10 per cent of the total ore of the stope.

Since 1983, the flat back blasthole stope method has been adopted (Fig. 3-8). With this method, the problems with the cohesiveness of the kaolinized ore were eliminated. It was determined that only 5 per cent of the ore was being left behind. In general, the stopes are set up on 22 metres widths (strike direction) with sublevels every 20 metres. A 4X4 m slot raise is driven down the hanging wall side of the stope. The slot raise is then used as an opening for mass blasting.

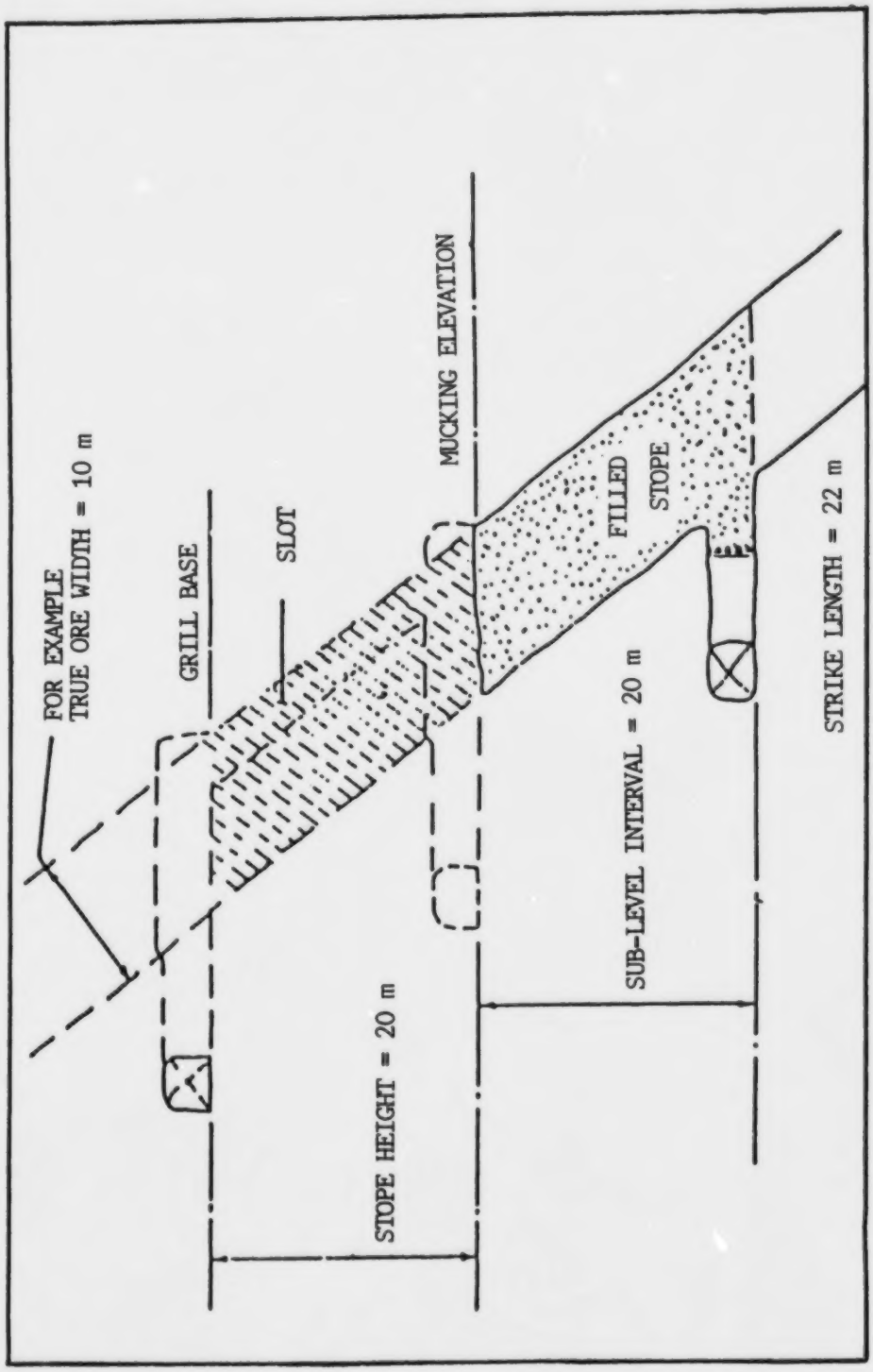


Figure 3-8 Typical Section Flatback Blasthole Stope

3.6. Ground Control Systems

Five major programs have been implemented to achieve ground control in the mine (Bryce, 1985). They are: bolting, backfilling, limitation on stope size, dewatering and ground monitoring.

3.6.1. Bolting

Extensive bolting patterns are used to ensure the stability of the openings at the mine. Depending on the ground conditions, all headings are bolted with extensive 1.5 m and 2.1 m long 1.6 cm diameter Stelco mechanical Rockbolts.

In the highly fractured ore zone, 1.9 m Swellex bolts are used to ensure a continuous contact along the full length of the bolt. Cement grouted cable bolts are employed extensively to hold in place the footwall fault ore, stope backs and the hanging wall of the stope immediately above the ore being mined, hence the problem with adopting a mining method to recovering such bolted surface crown pillars.

Screens are used in drifts passing through localized weak ground to minimize scaling and to improve mining safety.

3.6.2 Ground Monitoring

The mine is being monitored by means of the loading cells, extensometers, ground movement monitoring and ground subsidence measurements and piezometers.

Ground surface subsidence is being monitored by benchmarks set up around the mine property. The increased stresses due to mining operations are being measured by load cells. Underground rock movements are being determined by both extensometers and ground movement monitors.

The effectiveness of the dewatering system implemented in the mine is being monitored by piezometers.

3.6.3 Limitation of Stope Size

In general, the stopes beneath the surface crown pillar are mined out in 22 m widths along the strike direction. In areas where the orebody is wide (perpendicular to strike direction) and the ground is weak, the stope width is divided into halves along the strike direction. In the weakest ground (and also the widest and richest part of the pillar), the ore is mined between the 55 m and 70 m levels in 5.5 m wide panels. 11.0 metre wide panels are to be attempted in better ground.

Tentatively, a 7.5 m deep surface crown pillar is to be left in the ground. The possible future extraction of these surface crown pillars will depend on the experience gained in the mining of the crown pillars (Bryce, 1985).

3.6.4 Backfilling

A comprehensive backfilling program is implemented in the mined-out stopes. Once a stope has been mucked out, rockfill is dumped into the stope. Usually the inventory of unfilled stopes should not exceed two months production. Subsequent to initial rockfilling, fill fences are built across the bottom accesses of a drift. They are made of timber, screening, burlap and drain pipes. Once the fill fence is in place, cemented backfill is poured into the stope. Such cemented backfill consists of a 1:20 mixture of cement and tailings.

The water in the fill is drained off by means of a percolation tower. To ensure safety of the fence, excessive hydrostatic pressures which may develop behind the fence are monitored by means of a pressure gauge. The backfilling operation is halted once a pressure of 40 kPa is reached. Backfilling operations recommence when the porewater pressures within the backfills have dissipated.

The final mixture of the backfill has a ratio of 2:1 of rockfill to tailings (Bryce, 1985).

3.6.5. Dewatering

The initial high groundwater level which was present at the Mine site was lowered to the 180 m level through a drainage system which was set up by the mine (see Section 3.4). This reduction in water level increased the shear strength of the underground rock mass and also eliminated the possibility of a cave in such as that which occurred at the Belmoral Mine.

3.7 Cave-In 'A'

A cave-in occurred at stope number 4 in the fall of 1983. The material involved was estimated to be about 10,000 tons (Cutting, 1985). The cave-in started at the 100 m level and progressed upwards to approximately the 60 m level over a period of 6 to 8 months after mining of the stope.

Immediately after the cave-in, several geologists of the mine went underground to determine the shape of the cave. Detailed surveying of the shape of the cave could not be done because of the unstable conditions.

At the time of the cave-in, the ore was being mined with the diamond-shaped blasthole method. After the initial stope was blasted, the lower half of the diamond began to scale and created unsafe working conditions. This piece was then blasted, leaving a flat back stope with a span of approximately 50 metres across the strike of the orebody. After mucking, the stope was left unfilled. The approximate contours of the cave-in area are plotted in Fig. 3-9.

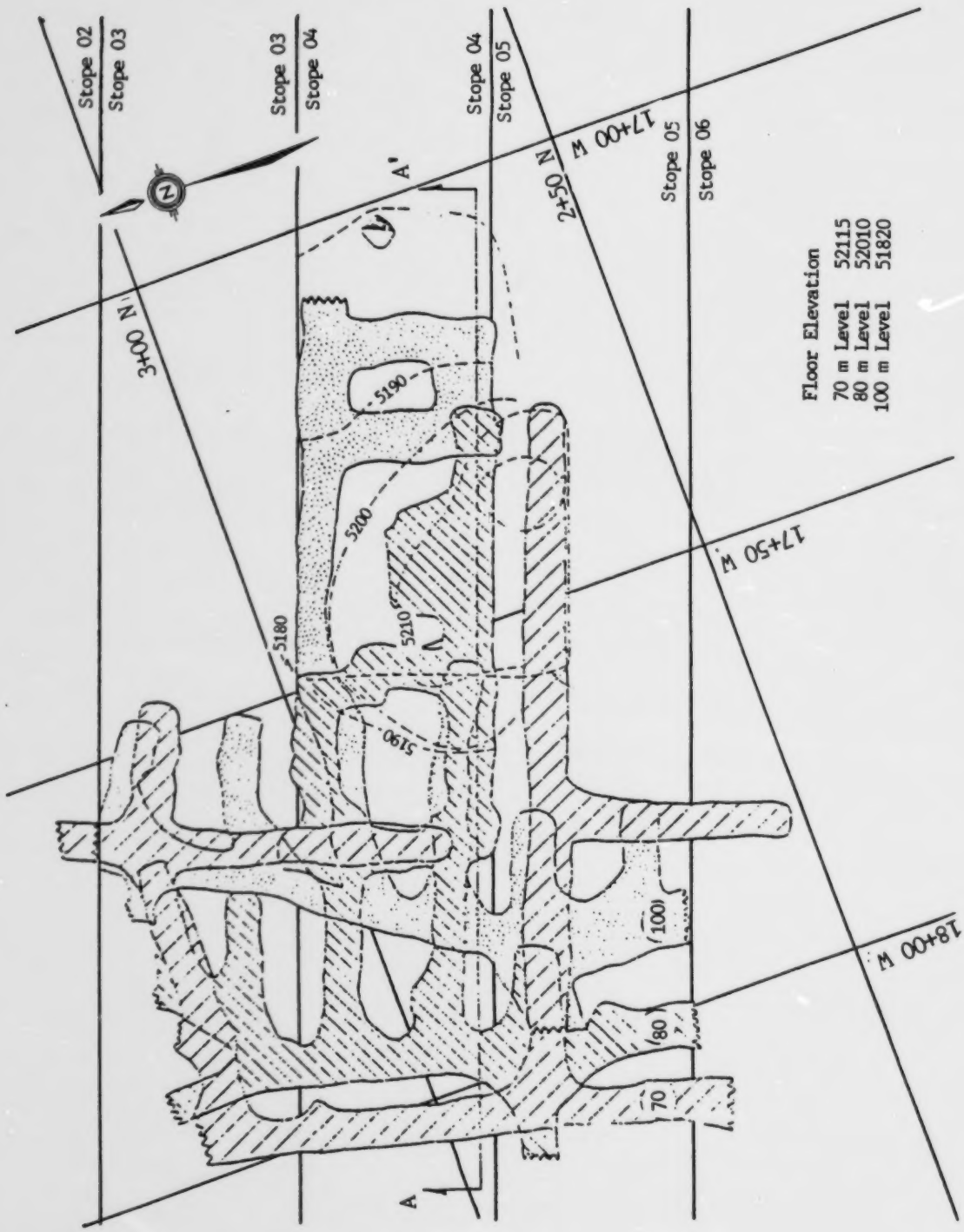


Figure 3-9 Underground development on 70, 80 and 100m levels showing Stope 4 Cave-in Contours in Plan View (Swan, 1985)

4.0 SURFACE CROWN PILLARS - A REVIEW

4.1 General

The surface crown pillar of the type which are being studied in this project is that class whose crown pillars are located in hard rock mines with near surface deposits (0 to 100 m). They are distinct from conventional deeper underground pillars.

A detailed literature review on surface crown pillars was conducted. However, limited information concerning this subject could be found. The major sources of information were case studies by Bétournay (1986, 1987) in which technical data from 132 surface crown pillars from 24 mines has been compiled. The roles of surface crown pillars have been identified by Bétournay (1986) as follows:

1. to protect workers from surface elements such as bodies of water, soil and precipitation;

2. to prevent interference with mining activity due to water inflow, surface subsidence and soil and rock sloughing.

3. to keep the extraction economical.

In general, the unique attributes of surface crown pillars are:

1. The designer must deal with unsuitable materials that cannot be substituted, and often faces dimensions and loads that cannot be altered.

2. Usually low values of lateral stresses occur near the surface which, given the altered and jointed characteristics of the rock mass, cannot be depended upon to contribute to stability. This implies that gravity and structure play the major roles in these structures.

4.2 Design Process

Because of the possible variation in geological and ore deposit characteristics at different mine sites, an integral design process has been suggested by Bétournay (1986) for the design of surface crown pillars. Briefly, this integral process consists of the following steps:

1. Identification of the deposit and regional rock characteristics
2. Geotechnical investigation
3. Establishing mining strategy and advance
4. Data analysis
5. Design methods and initial/final dimensioning
6. Mining activity
7. Monitoring
8. Back analysis
9. Recovery
10. Design evaluation

4.3 Design Methods

The various design methods available for the dimensioning of surface crown pillars are (1) empirical methods, (2) analytical methods and (3) numerical methods. The studies conducted by CANMET indicate that so far, most Canadian mines have dimensioned crown pillars based on empirical methods rather than using analytical methods based on in-situ rock mass characteristics and geotechnical data. For completeness of this Report, general descriptions of the methods for surface crown pillar dimensioning are presented in the next sections.

4.3.1 Empirical Methods

Based on rock mass classification systems , such as the NGI (Barton et. al., 1974) and the CSIR (Bieniawski, 1984), the size of underground openings can be estimated. By applying such an approach for the design of surface crown pillars, the span of the pillar can be determined. The thickness of the pillar cannot be obtained by this approach. In addition, the data used for the development of such classification systems are based mainly on observations which have been made in tunnels. In tunnel design, the ratio of length to width is very large, but in the case of mine openings, this ratio may be relatively small. Thus, the design of surface crown pillars based on this approach may be conservative at best, and at worst be seriously in error.

Another empirical approach for the design of surface crown pillars is based on local personal experience (CANMET, 1984, 1985). This method may not be totally reliable because of the possible variations and alterations in material properties from mine site to mine site.

4.3.2 Analytical Methods

In this type of method, the pillar is treated as a structural element. With various assumptions, the pillar is simplified and analyzed as an elastic fixed end supported beam, an elastic plate or a masonry arch (Hoek and Brown, 1980, Heyman, 1969).

In the case where the rock mass was altered or fractured, the tensile strength of the rock mass is assumed to be zero and plastic theory can be used to determine the dimensions of the pillar by treating the pillar either as a beam (Pender, 1985) or an arch (Heyman, 1969).

For well-bedded rock masses, design curves for roof and hanging walls based on the Voussoir beam and plate solution developed by Beer and Meek (1982) can be used.

4.3.3 Numerical Methods

Numerical modelling can be divided into two approachs: a continuum approach treating a rock mass as a continuum intersected by a number of discontinuities (Goodman, 1976) or a discontinuum approach (Cundall, 1971) regarding the mass as a group of independent blocks.

The selection of either method is based on the rock mass characteristics. Based on a geological description of the rock mass and from a comparison between the size of the opening and the spacing of discontinuities, the rock mass behavior model can be selected (Hoek, 1983).

With the determination of a rock mass behavior model, computer codes can be selected or developed for the design of the surface crown pillar.

4.4 Case Records

Two recent surveys conducted by CANMET (Betournay et. al., 1987) on surface crown pillars from 24 mines which are situated in the Canadian Shield, indicated that the ore deposits dip steeply and are generally either single or multiple veined. Considerable overburden generally caps the deposits. The rock mass is often altered and intersected by major discontinuities. A summary of the characteristics of surface crown pillars is presented in Table 4-1.

The design of these types of pillars which are given in Table 4-1 is based mostly on experience rather than on a scientific approach. A summary of the dimensioning methods is given in Table 4-2. Only four mines have used numerical modelling for the dimensioning of surface crown pillars. Of the 132 surface crown pillars recorded in the two studies, 27 are classified to be within competent ground, 86 in moderate ground and 19 in poor ground. A plot of the thickness/width ratio versus the number of surface crown pillars is given in Figure 4-1. This figure clearly indicates that regardless of the competence of the rock mass, the thickness to width ratio of the pillar is usually less than 5. In almost every case, rock bolting is used as a ground control measure. However, there are many openings which were neither backfilled nor monitored.

MINE ITEMS (3)	1	2	3•	4	5	6	7	8	9	10•	11	12	13	14	15	16	17	18	19	20	21	22	23
• BODY OF WATER (m)			3	(1)	-	-	-	7.6	-	N/A	N/A	-	20	-	-	-	-	-	11	-	-	-	13
• OVERBURDEN (m)	(2)	(2)	27	36	4	15	5	20	17	16	20	15	3	5	30	9	1.5	(2)	5	-	45		19
substantial clay deposits			"	"				"	"		"	"		"								N/A	"
• FORM OF THE DEPOSIT																							
- tabular	"				"																		
- single vein			"	"				"	"	"	"		"		"		"				"		"
- multiple veins		"				"	"							"				"					
- mass												"				"			"	"		"	
• Pronounced alterations	"			"	"			"								"							
• Walls of low competence	"	"	"		"			"						"			"		"		"		"
• Walls of high competence						"	"			"			"	"	"	"			N/A	N/A	N/A	N/A	"
DIP (degrees)	70	70	65	45	72	80	80	90	45	70	80	85	45	85	75	75	33	70	N/A	N/A	N/A	N/A	
IMPORTANT FAULT(S)		"	"	"		"		"	N/A	"	"	"	"		"	"		"	70	60	50	75	30
NUMBER OF WELL																			N/A	N/A	"	"	N/A
DEFINED JOINT FAMILIES	N/A	2	3	N/A	2	2	3	N/A	N/A	N/A	N/A	N/A	N/A	2	N/A	N/A	1	3	N/A	N/A		N/A	N/A
MAIN MINING METHOD																							
- stope and pillars					"	"	"		"								"						
- shrinkage stoping			"																				
- cut-and-fill				"				"		"		"		"	N/A	"			"		"		"
- blasthole stoping	"	"							"		"		"		N/A	"		"	"		"		"
• USE OF FILL		"	"					"	"		"	"	"	"									

N/A not retrieved, or not available
(1) drained
(2) overburden removed
(3) average, applicable to surface crown pillars
(4) pillar(s) separating open pit from underground opening

Table 4-1 Basic Characteristics of Surface Crown Pillars of Hard Rock Mines (Bétournay et.al., 1987)

DIMENSIONING METHOD	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
- elastic analytical calculation	3	0	0	0	0	0	0	0	0	0	0	0	0	0	-	0	0	0	0	0	0	0	0	-
- plastic analytical calculation	0	0	0	0	0	0	0	0	0	1	0	0	0	0	-	0	0	0	0	0	0	0	0	-
- numerical modelling	0	0	3	0	0	0	0	0	0	0	0	0	0	0	-	0	0	0	0	3	3	0	3	-
- probalistic	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-	0	0	0	0	0	0	0	0	-
- empirical	0	0	0	0	0	1	0	0	1	0	1	0	1	0	-	0	0	3	3	0	3	0	0	3

-	not applicable
0	no study
1	limited study
2	in-depth study
3	complete study
N/A	not available; not retrieved

Table 4-2 Dimensioning of pillars (Bétournay et.al., 1987)

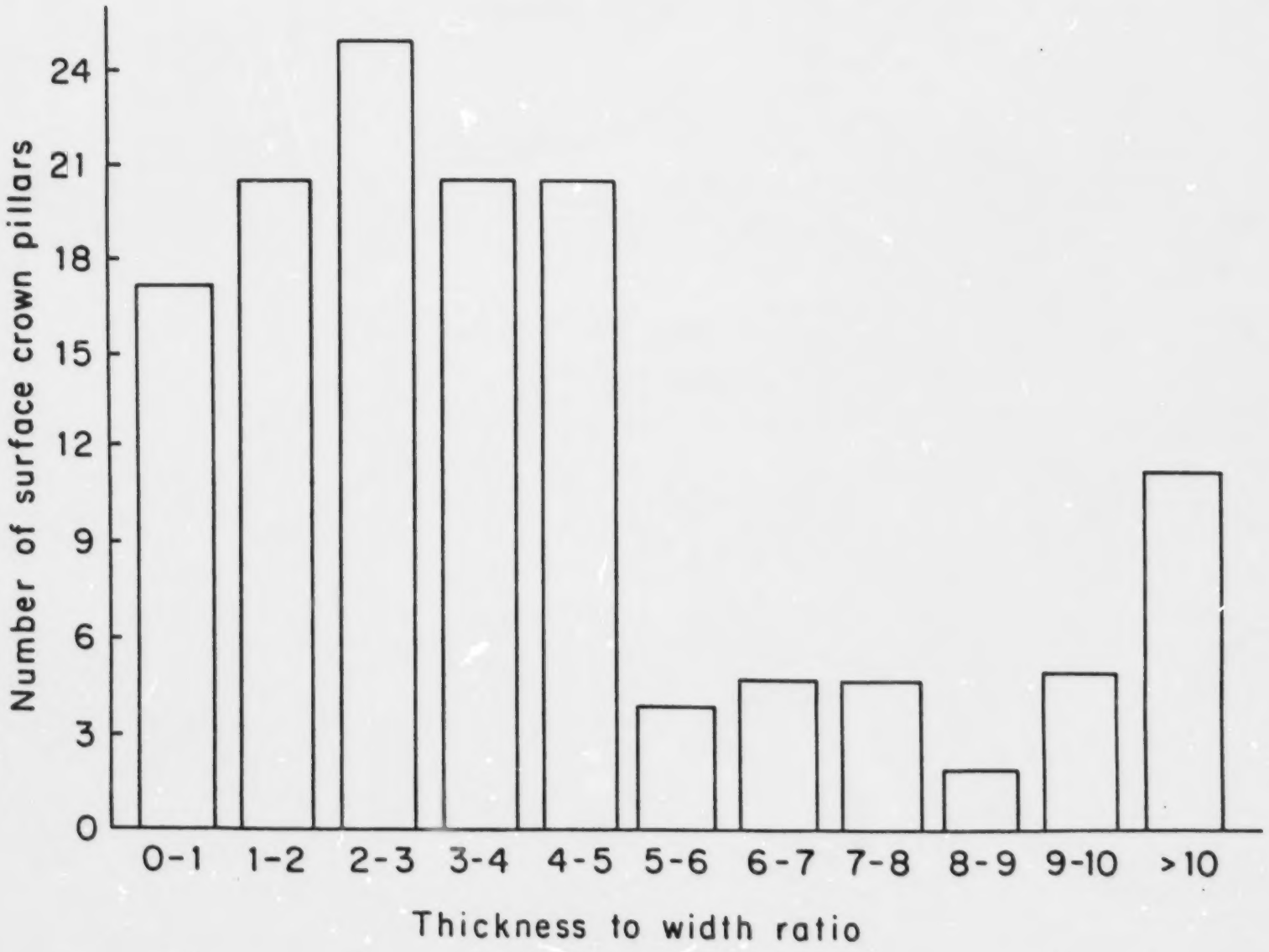


Figure 4-1 Surface Crown Pillars: Thickness / Width ratio, 132 pillars (Bétournay et. al., 1987)

5.0 LES MINES SELBAIE SURFACE CROWN PILLAR - ROCK MASS BEHAVIOR MODEL

In the selection of an appropriate rock mass behavior model for the surface crown pillar at Les Mines Selbaie, the rock mass characteristics, and the strength and deformation characteristics have to be defined. A summary of the rock mass characteristics is defined by Hoek (1983) and is presented in Fig. 5-1. In the following sections, the different factors to be considered in the selection of a rock mass behavior model for the surface crown pillar at Les Mines Selbaie are examined. These factors are the scale factor, the volumetric ratio and the failure mode.

5.1 Scale Factor

The scale factor in this report is defined as the ratio between the size of the surface crown pillar being considered and the spacing of discontinuities in the rock mass (eg. block size). A schematic representation (Fig. 5-2) of such an effect is given by Hoek (1983). Based on the geological description of the rock mass and the dimensions of the surface crown pillars being designed, the rock mass characteristcs of the pillar (Fig. 5-1) can be determined. As shown in Fig. 5-1, different rock masses will have different characteristics and these in turn will have a significant effect on the selection of the rock mass behavior model to be used in the study of surface crown pillars.

	Description	Strength characteristics	Strength testing	Theoretical considerations
	Hard intact rock	Brittle elastic and generally isotropic	Triaxial testing of core specimens in laboratory relatively simple and inexpensive and results usually reliable	Theoretical behaviour of isotropic elastic brittle rock adequately understood for most practical applications
	Intact rock with single inclined discontinuity	Highly anisotropic depending on shear strength and inclination of discontinuity	Triaxial testing of core with inclined joints difficult and expensive but results reliable Direct shear testing of joints simple and inexpensive but results require careful interpretation	Theoretical behaviour of individual joints and of schistose rock adequately understood for most practical applications
	Massive rock with a few sets of discontinuities	Anisotropic, depending on number, shear strength and continuity of discontinuities	Laboratory testing very difficult because of sample disturbance and equipment size limitations	Behaviour of jointed rock poorly understood because of complex interaction of interlocking blocks
	Heavily jointed rock	Reasonably isotropic Highly dilatant at low normal stress levels with particle breakage at high normal stress	Triaxial testing of undisturbed core samples extremely difficult due to sample disturbance and preparation problems	Behaviour of heavily jointed rock very poorly understood because of interaction of interlocking angular pieces
	Compacted rockfill	Reasonably isotropic Less dilatant and lower shear strength than in situ jointed rock but overall behaviour generally similar	Triaxial testing simple but expensive because of large equipment size required to accommodate representative samples	Behaviour of compacted rockfill reasonably well understood from soil mechanics studies on granular materials
	Loose waste rock	Poor compaction and grading allow particle rotation and movement resulting in mobility of waste rock dumps	Triaxial or direct shear testing relatively simple but expensive because of large equipment size required	Behaviour of waste rock adequately understood for most applications

Figure 5-1 Summary of Range of Rock Mass Characteristics (Hoek, 1983)

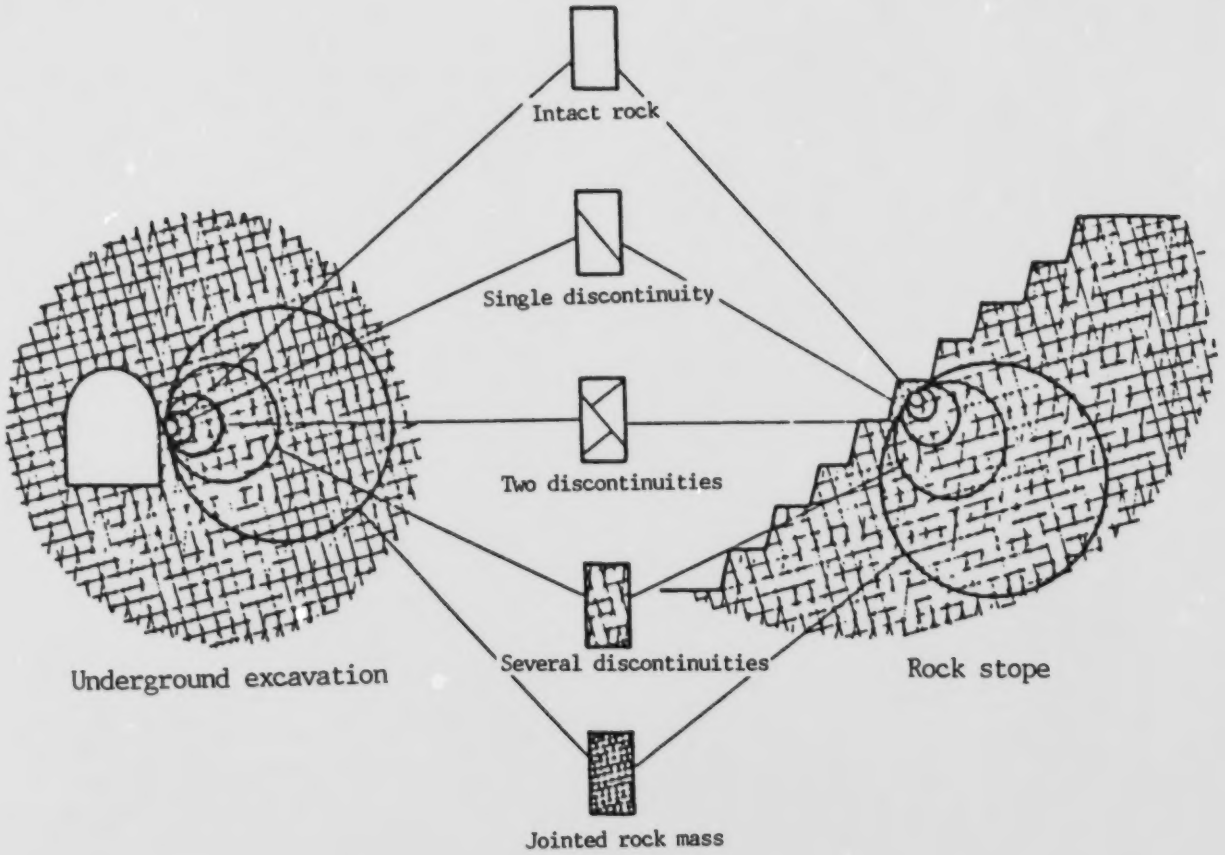


Figure 5-2 Simplified representation of the influence of scale on the type of rock mass behaviour model which should be used in designing underground excavations or rock slopes (Hoek, 1983)

5.2 Volumetric Ratio

In situations where an alteration process weakens the rock mass, such as at Les Mines Selbaie, the mechanical characteristics of the rock mass, as well as the composition change. Here, the volumetric ratio of an altered rock mass refers to the ratio of the volume of the altered material to the total volume of the material.

The volumetric ratio of an altered rock mass is one of the most important factors to be considered in the selection of a rock mass behavior model for surface crown pillar design.

When the alteration process is completed, the rock mass may be completely converted to a clay. Depending on the stress level, the material can be treated as an elastic medium or an elastoplastic medium.

When the alteration process is limited, the rock mass may be composed of intact rock pieces embedded in a matrix of the altered material. In such a case, the medium can be treated as a blocky ground with clay-filled joints.

Under different alteration conditions, the resulting size of the remaining rock pieces and their packing may be very different although their volumetric ratio may be the same. No published data has been found on the volumetric ratio of rock mass having different sizes and shapes of rock pieces and under different packing conditions.

By analogy, for a granular soil, one can envision that the solid grains are the remnant rock pieces left after the alteration process with air voids filled with the altered material. Thus a comparison is possible between the porosity of a soil mass (volume of voids as a per cent of total volume) and the volumetric ratio of an altered rock mass. Table 5-1 provides data on the volumetric ratio of altered rock masses with different size and shape of rock pieces under different packing conditions. The volumetric ratio for a loosely packed soil ranges from 44 to 52 per cent and for a densely packed soil from 12 to 33 per cent.

The packing of the rock pieces has a very significant effect on the selection of a rock mass behaviour model. For a specific packing of the rock pieces, the number of contacts between the rock pieces decreases with an increase in the volumetric ratio. With further increases in the volumetric ratio, the rock pieces may become suspended in the clay matrix , with zero or minimal contact between them.

Description	Void Ratio		Porosity (%)		Dry Unit Weight (kN/m^3)	
	e_{max}	e_{min}	n_{max}	n_{min}	$\gamma_{d\,min}$	$\gamma_{d\,max}$
Uniform spheres	0.92	0.35	47.6	26.0	—	—
Standard Ottawa sand	0.80	0.50	44	33	14.5	17.3
Clean uniform sand	1.0	0.40	50	29	13.0	18.5
Uniform inorganic silt	1.1	0.40	52	29	12.6	18.5
Silty sand	0.90	0.30	47	23	13.7	20.0
Fine to coarse sand	0.95	0.20	49	17	13.4	21.7
Micaceous sand	1.2	0.40	55	29	11.9	18.9
Silty sand and gravel	0.85	0.14	46	12	14.0	22.9

Table 5-1 Maximum and Minimum Densities for Granular Soils (Hough, 1957)

5.3 Failure Mode

For an altered rock mass, the failure mode of the surface crown pillar can have a significant effect on the selection of a rock mass behavior model.

For compressive or shear type failure, the behavior of jointed rock masses is poorly understood because of the interlocking of angular pieces. During deformation at low normal stresses, the material will dilate. At high normal stresses, particle breakage will occur.

When a severely altered jointed rock mass is subjected to tensile stresses the rock pieces are pulled away from each other. In this case, the strength of the rock mass is equal to the tensile strength of the weak altered material. Thus in tension, the failure mode of a severely altered rock mass could be similar to a clay medium except in local areas where the failure surface is deflected by local rock pieces.

5.4 Les Mines Selbaie Surface Crown Pillar Model

The effect of various factors on the selection of a rock mass behavior model for surface crown pillars was discussed in the previous section. The logical steps for the selection of a rock mass behavior model involve the determination of the scale factor, volumetric ratio of the material and then modifying the choice, if necessary with information on the anticipated failure modes.

A detailed survey of discontinuity orientation, spacing and length at the 55, 60 and 70 metre levels of the Mine was conducted by CANMET at Les Mines Selbaie (Arjang, 1984). The results of this study are summarized in Table 5-2.

In general, three major discontinuity sets were mapped. In the ore zone, the mean spacing was determined to be 0.59 to 0.72 metres in the south and north sections respectively. The result of a field survey with needle penetrometer along the 05-2-12 drift by CANMET (Swan, 1985) is shown in Fig. 5-3. This figure indicates that the soft zones (kaolinized material) were interbedded in the host rock blocks. The maximum block size estimated from Fig. 5-3 is approximately 0.75 m. This result is compatible with the result obtained from joint mapping. By comparing the possible block size of 0.5 to 0.75 m to the width of the stope size of 22 m, the rock mass characteristics may be classified as a heavily jointed rock mass.

Location		Scanline (m)	No. of Observations	Mean Spacing (m)	λ	RQD %
H/W		30	42	0.71	1.40	99.1
O/B	N	37	51	0.72	1.37	99.1
	S	143	85	0.59	1.68	98.7
F/W	N	100	134	0.74	1.34	99.1
	S	208	273	0.76	1.31	99.2

Table 5-2 Estimated RQD values from discontinuities spacing (Arjang, 1984)

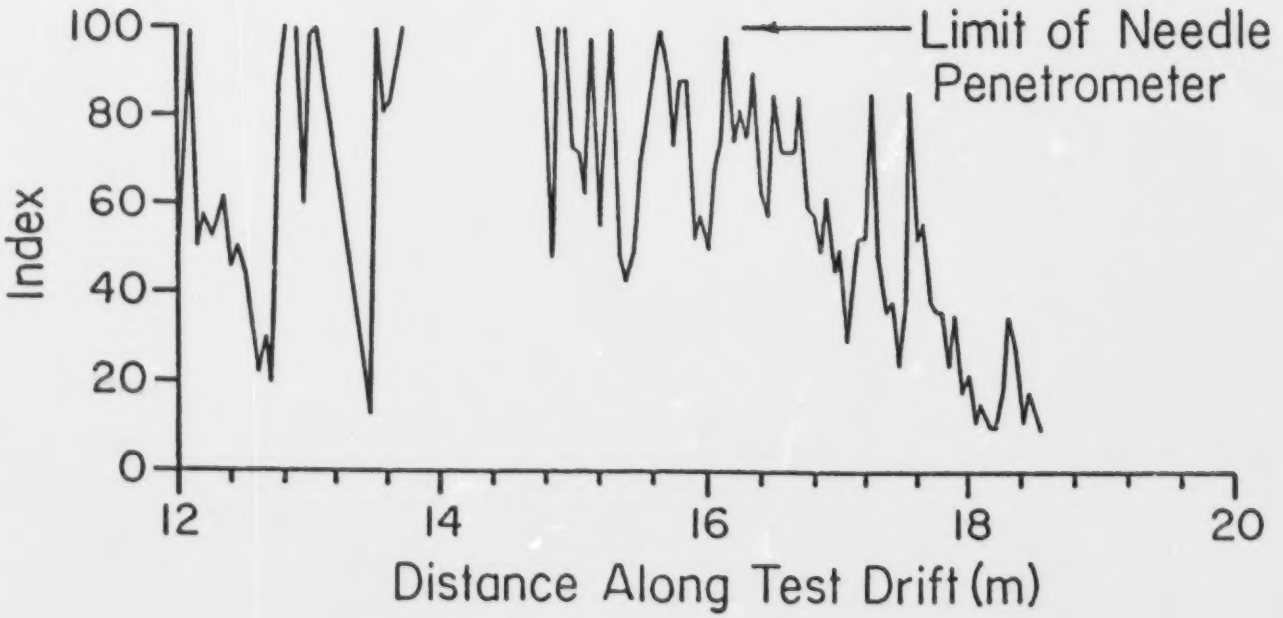


Figure 5-3 Results from needle penetrometer survey of 05-2-12 drift (Swan, 1985)

The volumetric ratio at the surface crown pillar was determined by a scanline survey based on a classification system developed in Phase I of this study and is about 30 per cent. This volumetric ratio value is much higher than the 15 per cent value reported earlier by other workers (Swan, 1985). This is possibly due to the increases in alteration of the orebody with an increase in mine elevation. The shape of the altered rock blocks at Les Mines Selbaie has not been characterized.

If the rock pieces are round and arranged in a loose configuration, the required volumetric ratio for the contact between the rock pieces to be separated are 44 to 52 per cent (Table 5-1) which is much larger than the 30 per cent estimate for Les Mines Selbaie. This means that the rock mass behaviour of the surface crown pillar at Les Mines Selbaie will be that of a jointed rock mass.

If the rock mass consists of densely packed round rock pieces, the required volumetric ratio for the rock pieces to be in contact is 12 to 33 per cent which indicates that the rock pieces at Selbaie Mine are probably "floating" in a matrix of the kaolinized material. Thus, if the rock mass is densely packed the surface crown pillar could be treated as an elastoplastic medium.

The thickness of the kaolinized material has been calculated based on the assumption that the rock pieces had a cubical shape. Based on a specific volumetric ratio, the thickness of an altered material between cubical rock pieces, arranged in a cubic configuration, is presented in Table 5-3.

CUBICAL BLOCK SIZE (m)	THICKNESS OF ALTERED MATERIAL (cm)	
	VOLUMETRIC RATIO 15%	VOLUMETRIC RATIO 30%
0.75	8	19
0.60	6.6	15

Table 5-3 Thickness of Altered Material vs. Block Size and Volumetric Ratio

From an elastic analysis of cave-in 'A' which is based on the elastic modulus determined by field pressuremeter tests, the predicted average displacement in the rock mass is about 5 cm, which is less than the values listed in Table 5-3. This indicates that if the rock pieces at Les Mines Selbaie are cubic in shape and arranged in a cubical configuration then the pillar can be treated as an elastoplastic medium consisting of kaolinized material, since the rock blocks are unlikely to come in contact during deformation.

Because of the unknowns involved with respect to the shape, size, distribution and packing of the rock pieces of the material at the surface crown pillars at Les Mines Selbaie , no definite conclusion can be drawn on a rock mass behaviour model for the pillar.

The possible failure mode of the surface crown pillar at Les Mines Selbaie was studied using a Base Friction Model (Goodman, 1976). A mixture of silica sand, flour, vegetable oil and clean gravel (of various selected sizes) was used to represent the soil-like rock mass. Several variations in the proportions of these constituents were used in a number of trials to obtain a "feel" for the correct consititutive mix.

A tensile type failure due to gravitational force was observed (Figure 5-4). The final shape of the opening was an arch, which is similar to observations made in cave-in 'A'. These findings favored the selection of an elastoplastic rock mass behavior model for the analysis of the surface crown pillar.

Owing to the uncertainty involved in the shape and size distribution and the nature of the rock pieces, the shear strength of the rock mass was determined for both a kaolinized material and a jointed rock with kaolin infilling.

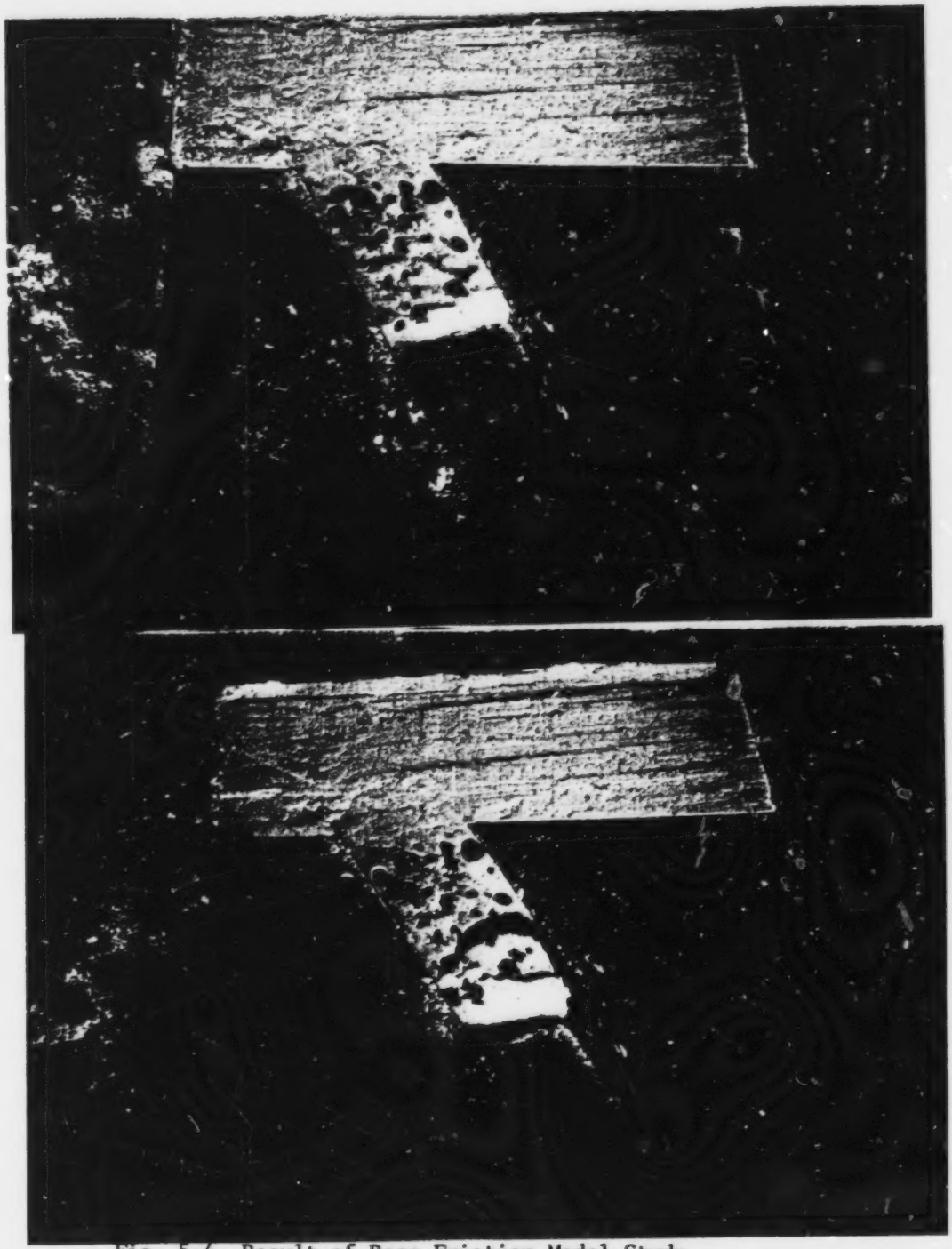


Fig. 5-4 Result of Base Friction Model Study

The kaolinized material of the mine has a plasticity index of 7 per cent. From Fig. 5-5, the angle of friction of the soil was estimated to be 33.5 degrees.

The shear strength characteristic of a jointed rock mass with infilling can be determined from an empirical chart developed by Hoek (1983). With the NGI or CSIR rock mass rating and the rock type, the m and s value of Hoek and Brown(1980) can be determined (Table 5-4).

The rock mass at Les Mines Selbaie has been characterized by both NGI Tunnelling Quality Index and CSIR Geomechanics classification systems. The result of the study (Arjang, 1984) is presented in Table 5-5. The suggested overall assessment of the rock mass strength of the orebody in the south section of the mine is poor.

With a Q-index equal to 3.25 and a CSIR Geomechanics classification rating of 40, the m and s values, as determined from Table 5-4, are 0.31 and 0.0001 respectively. With these values, a compressive strength of 33 MPa for the rock pieces, an average overburden density of 26.5 kN/m3 and a 50 m thickness of overburden, the ϕ' and c' values were determined to be 33.2 degrees and 0.046 MPa respectively.

The shear strength parameter obtained from the jointed rock mass behavior model is compatible to that of the kaolinized material.

From the above study, a kaolinized soil model is proposed as the behavior model for the surface crown pillar at Les Mines Selbaie.

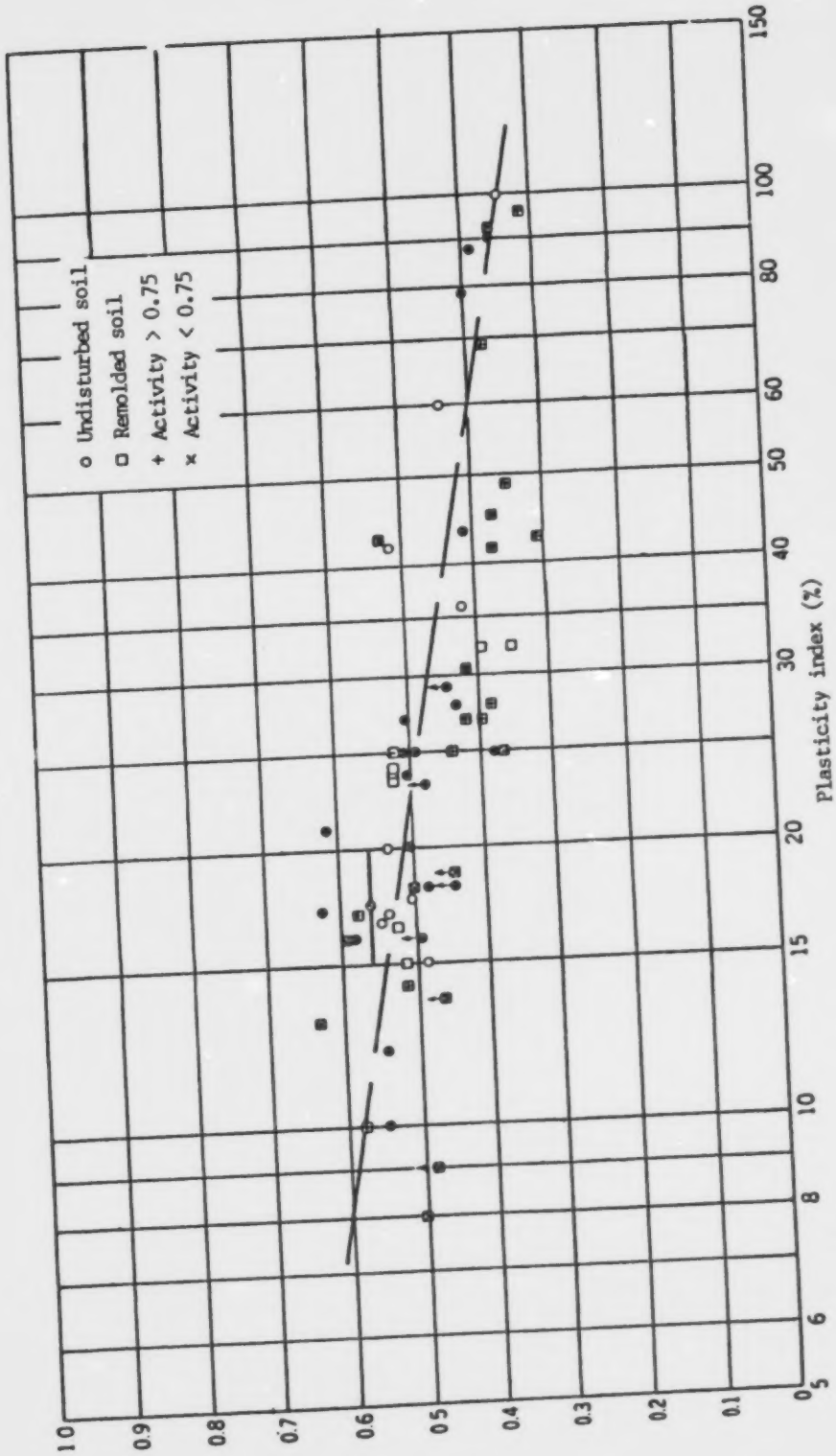


Figure 5-5 Relationship between Sin Ø and plasticity index for normally consolidated soils (Kenney, 1959)

Emperical Failure Criterion $\sigma_1 = \sigma_3 + (m\sigma_c\sigma_3 + s\sigma_c^2)^{1/2}$ σ_1 = major principal stress σ_3 = minor principal stress σ_c = uniaxial compressive strength of intact rock m,s=emperical constants	Carbonate Rock Well developed crystal cleavage eg.dolomite, limestone, marble	Lithified argillaceous rocks, eg. mudstone, siltstone, shale & slate (Tested normal to cleavage)	Arenaceous rocks with strong crystalss & poorly developed crytal cleavage eg. sandstone & quartz	Fine grained polyminerallic igneous crystalline rocks, eg. andesite,dolerite diabase & rhyolite	Coarse grained polymineralic igneous & metamorphic crystalline rocks, eg. amphibolite, gabbro, granite, norite & quartz-diorite
Intact rock samples Lab size free from pre-existing fractures CSIR rating 100 NGI rating 500	m = 7 s = 1	m = 10 s = 1	m = 15 s = 1	m = 17 s = 1	m = 25 s = 1
Very good quality rock mass Tightly interlocking undisturbed rock with rough unweathered joints spaced at 1-3 m CSIR rating 85 NGI rating 100	m = 3.5 s = 0.1	m = 5 s = 0.1	m = 7.5 s = 0.1	m = 8.5 s = 0.1	m = 12.5 s = 0.1
Good quality rock mass fresh to slightly weathered rock, slightly disturbed with joints spaced at 1-3 m CSIR rating 65 NGI rating 10	m = 0.7 s =0.004	m = 1 s =0.004	m = 1.5 s =0.004	m = 1.7 s =0.004	m = 2.5 s =0.004
Fair quality rock mass several sets of moderately weathered joints spaced at 0.3-1.0 m CSIR rating 44 NGI rating 1	m = 0.14 s =0.001	m = 0.2 s =0.001	m = 0.3 s =0.001	m = 0.34 s =0.001	m = 0.5 s =0.001
Poor quality rock mass numerous weathered joints spaced at 30-500 mm with some gouge. Clean compacted rockfill CSIR rating 23 NGI rating 0.1	m = 0.04 s=.00001	m = 0.05 s=.00001	m = 0.08 s=.00001	m = 0.09 s=.00001	m = 0.13 s=.00001
Very poor quality rock mass Numerous heavily weathered joints spaced at 50 mm with gouge. Waste rock CSIR rating 3 NGI rating 0.01	m =0.007 s = 0	m = 0.01 s = 0	m= 0.015 s = 0	m =0.017 s = 0	m= 0.025 s = 0

CSIR - Commonwealth Scientific & Industrial Organization (Bieniawski, 1974)
NGI - Norwegian Geotechnical Institute (Barton et al.,1974)

Table 5-4 Approximate relationship between rock mass quality and material constants (Hoek, 1983)

Location	Q-System		Geomechanics Classification		RMR	Comments
	Q-Index	R.M. Class	Rating	R.M. Class		
H/W	14.85	'Good'	48(52)*	'Fair'	54	Equation: RMR = 9 Log Q + 44 defined the relationship between the two systems for correlation purposes.
O/B N	4.36	'Fair'	45(49)	'Fair'	49	
O/B S	3.25	'Poor'	40(44)	'Poor'	48	
F/W N	6.54	'Fair'	46(50)	'Fair'	51	
F/W S	4.36	'Fair'	43(47)	'Fair'	49	
Fault zone	0.06	'Extremely poor'	16	'Very poor'	33	

* Over-estimated rock mass ratings are given in brackets.

Table 5-5 Comparison of rock mass characteristics in Q-system and Geomechanics Classification (Arjang, 1984)

The kaolinized soil has an angle of friction of 33.5 degrees and an apparent cohesion of 0.046 MPa. This selected rock mass behavior model was used in a computer code for the analysis of cave-in 'A'. The kaolinized soil model will be validated and calibrated by comparing the analytical results to the field measurements made in cave-in 'A' in the next section.

6.0 ANALYSIS

6.1 Computer Code Selection

The computer code, ADINA (Automatic Dynamic Incremental Nonlinear Analysis), is selected for the analysis of cave-in 'A' as reported by the mine. ADINA code is a multi-dimensional finite element computer program. It can handle the rock mass behavior model selected in the last chapter through its Mohr-Coulomb type elements. The reason for the selection of the ADINA code instead of other compatible computer codes is that ADINA has a 'birth and death' option for its elements. This option provides great convenience in modelling, especially in cases where tensile failure occurs and where these failed elements have to be removed from the element mesh. Instead of constructing a new element mesh and preparing all the input data again, such failed elements can be specified as unborn elements in the next cycle of the calculations.

6.2 Back Analysis of Cave-In 'A'

Initially a two-dimensional analysis was performed to simulate cave-in 'A' at the Mine. Subsequently, this two-dimensional analysis was discontinued due to recognition of the significance of three-dimensional effects.

6.2.1 Material Properties

The shear strength properties of the orebody were determined to have an angle of friction of 33.5 degrees and a cohesion of 0.042 MPa. The unit weight of the material was taken as 25.6 kN/m3. No test data on the tensile strength of the kaolinized material is available because of the difficulties involved in the preparation of specimens for testing. An empirical value based on one-tenth of the compressive strength was used, which is equal to 0.06 MPa (Fig. 3-3).

Because of the variation in the distribution of the materials, the deformation modulii determined by field pressuremeter tests (Swan, 1985) and dilatometer test (Bétournay, 1987) are very different. The lower value of the deformation modulus determined by Bétournay is probably the result of local soft zones.

Owning to the unknown distribution of the kaolinized material, an average deformation modulus is estimated based on the composite formula (Swan, 1985) and the test results from Swan(1985) and Bétournay(1987). With a volumetric ratio of 30 per cent, the calculated deformation modulus is 0.6 GPa.

The properties of the footwall, hanging wall and the overburden are listed in Table 6-1.

MATERIAL	E GPa	ν	γ MN/m^3
ROCK	40.000	0.20	0.026
OVERBURDEN	0.039	0.40	0.022
BACKFILL (Cement, Rockfill and Tailings)	2.000	0.25	0.023

Table 6-1 Material Properties Used in Analyses of Cave-in 'A'

6.2.2 Field Stress

The field stress values determined by CANMET (Arjang, 1984) are listed in Table 3-4. The horizontal to vertical stress ratio across the strike direction as determined at the 70 m level is unusually low, much less than the value expected where considering those values obtained at the 180 and 290 m levels. This may be due to the occurrence of kaolinized material above the 100 m level (Swan, 1985). The large value of horizontal to vertical stress ratio along the strike direction is probably caused by stress concentrations due to the shape of the orebody.

Based on the large difference between the deformation modulus of the host rock, the orebody at the surface crown pillar and the footwall and hanging wall faults, the in-situ stress in the surface crown pillar were assumed to be those obtained in a k_o(at rest) condition.

6.2.3 Geometry

The shape of the orebody and the location of stope number 4 where the cave-in occurred are shown in Fig. 6-1. In the back analysis, the shape of the orebody was modified in order to simplify the geometry and reduce the number of elements required.

The geometry of the orebody used for this study and the location of stope no. 4 are shown in the same figure. With such a simplified geometry, a total of 600 elemets was used for the analysis.

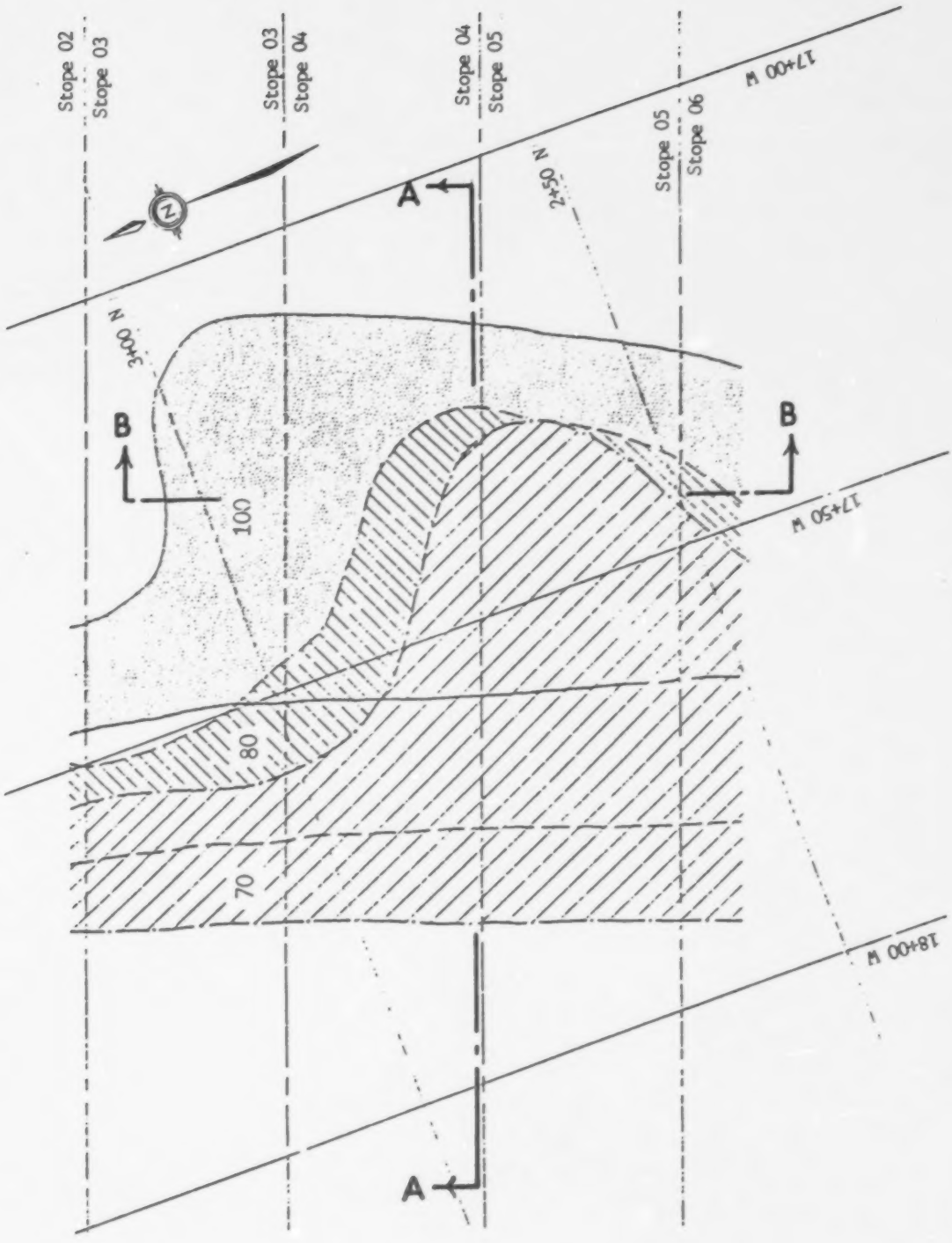


Figure 6-1a Plan view of Orebody, 70 /80/ 100m levels

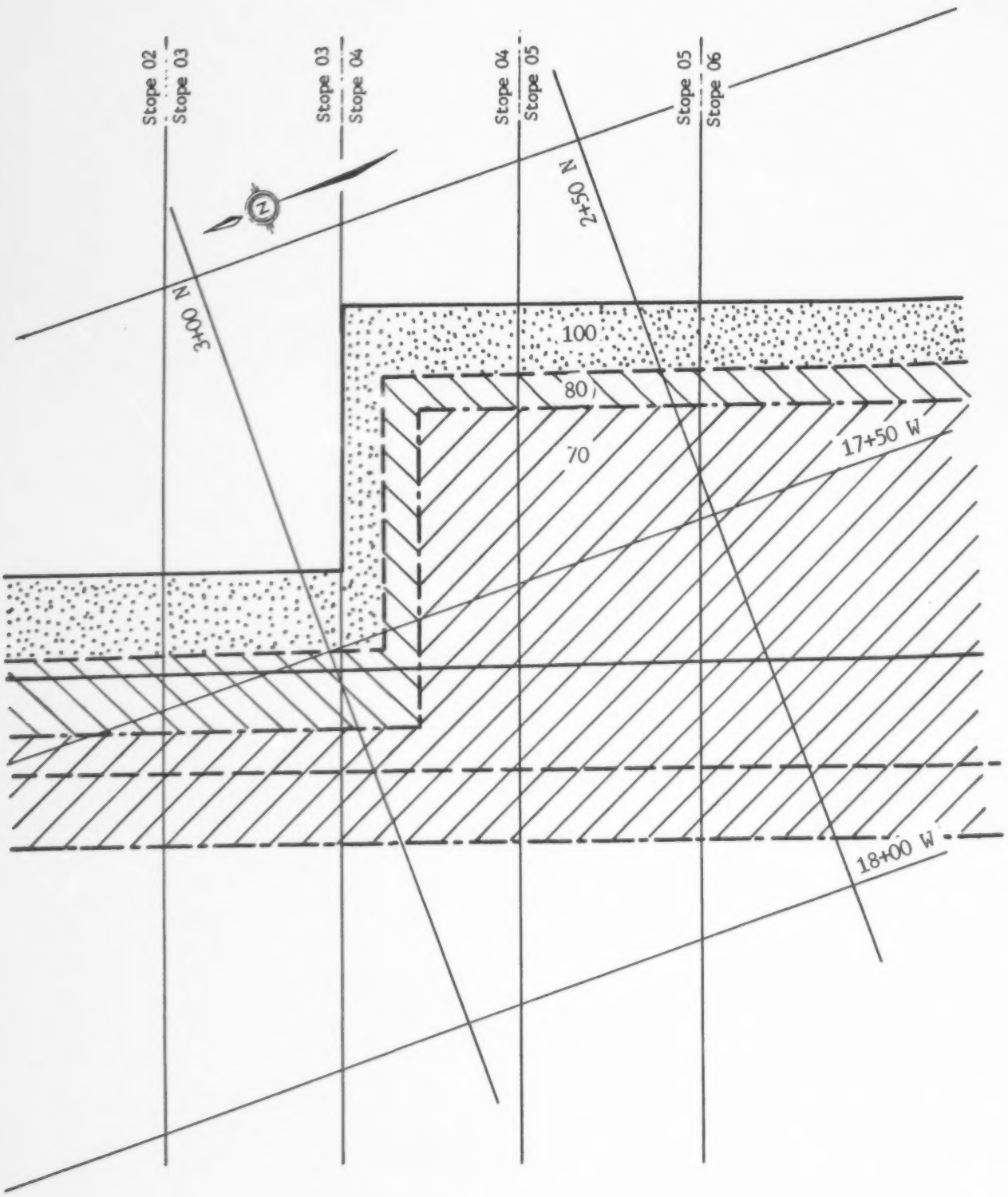


Figure 6-1b Plan View of Orebody, 70 /80/ 100m levels, -- Model

A schematic diagram of the layout of stope no. 4, the backfill , the orebody and the rock are shown in Fig. 6-2. In addition, the boundary conditions assumed in this study are depicted.

6.2.4 Method of Analysis

During analysis, when extensive tensile failures occurred, the computer program became numerically unstable. In order to avoid such a problem, the analysis was conducted such that the gravitational forces were applied in 10 computational steps. With the advance in these steps, when the calculation became unstable (for example, no convergence), the analysis was halted and all the tensile failed elements were eliminated by changing the status of those failed elements into unborn elements. The analysis was then continued until full gravity load was applied.

6.3 Results of Analysis

6.3.1 Predicted vs. Actual Comparisons

Because of the complexity of the three-dimensional stress field, no plot of the stress field was made. Instead the results of the final failure surface has been plotted against the field cave-in surface. The cross-sections, A-A and B-B (ref. Fig. 6-1), for the failure surfaces for both the field record and analytical results are plotted in Figs. 6-3 and 6-4.

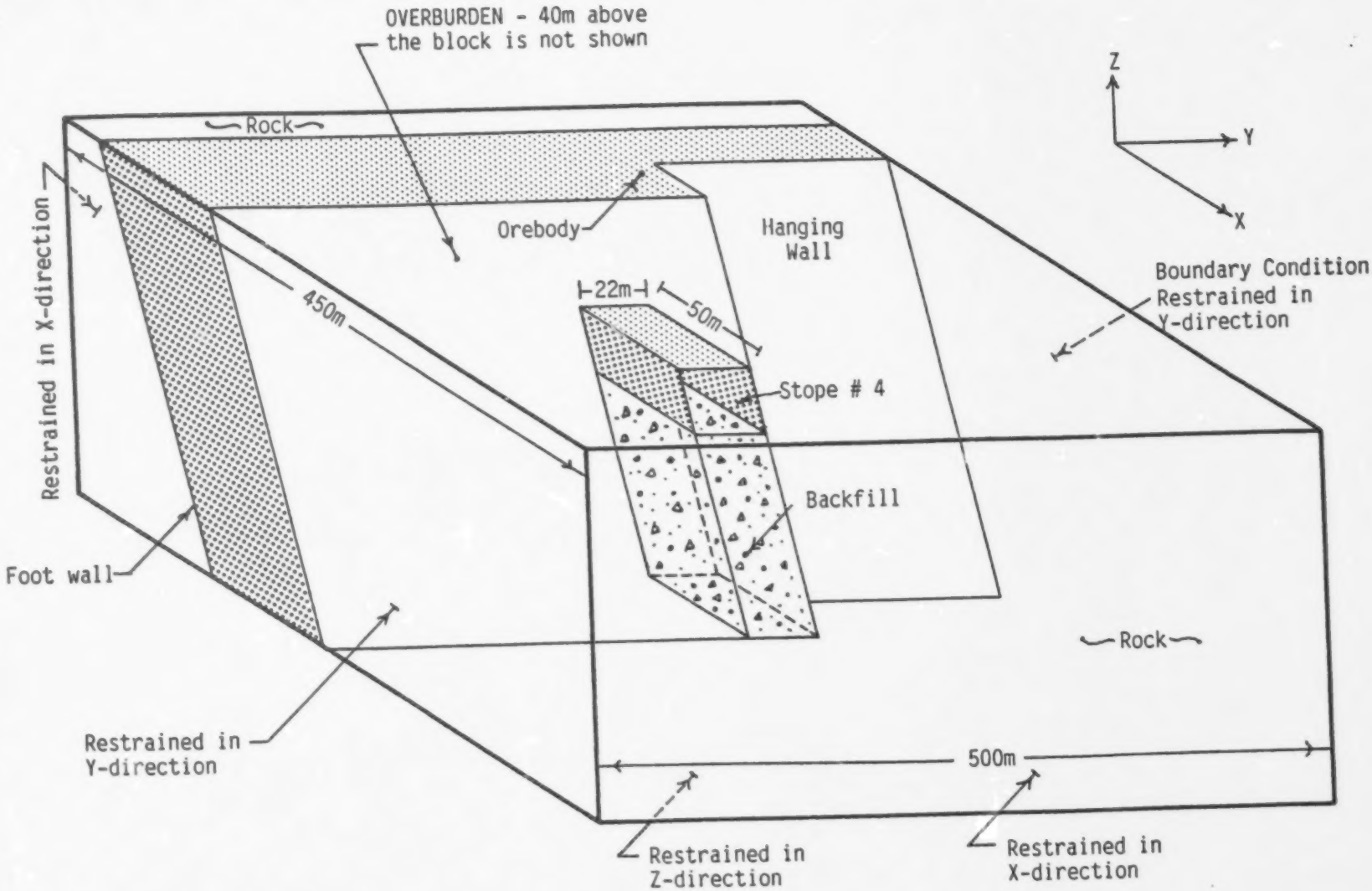


Figure 6-2 Schematic Diagram of the Layout of the Model Used for the back calculation of Cave-in 'A'

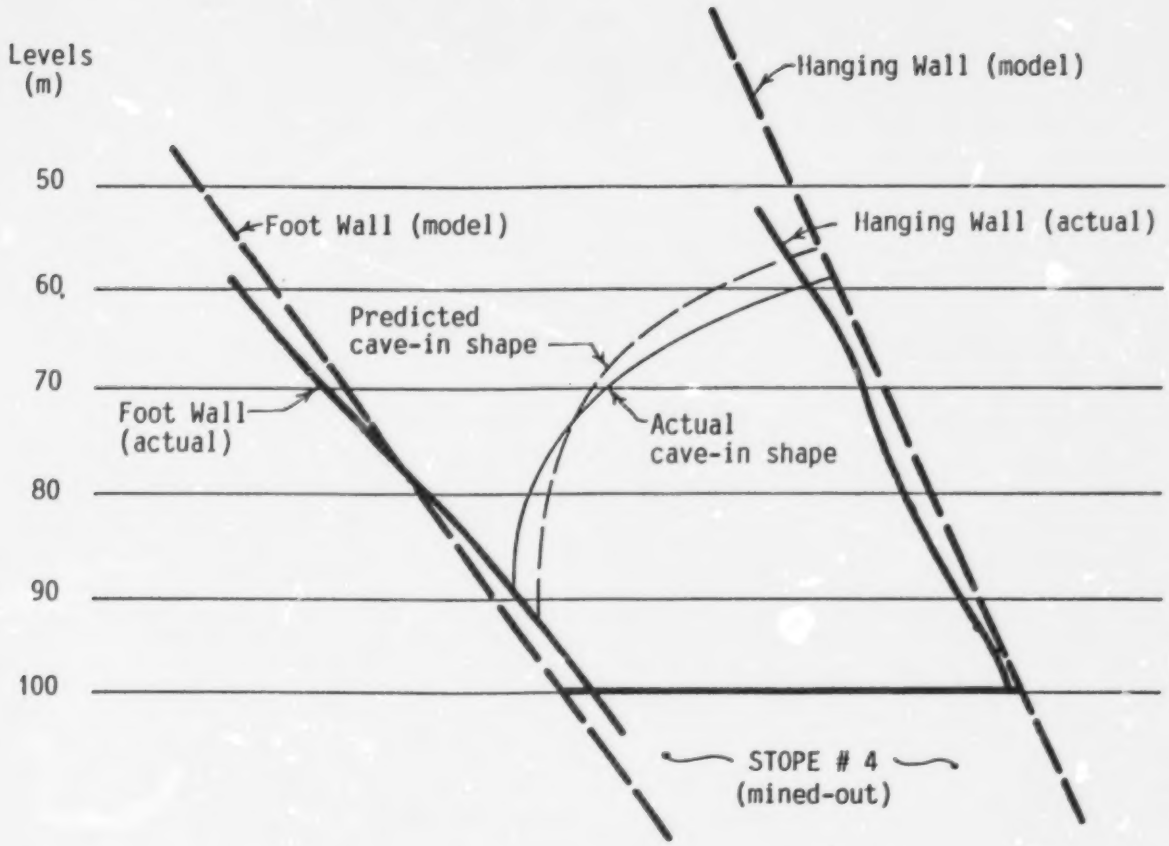


Figure 6-3 The Predicted and Actual Failure Surface - Cross Section A-A (looking along Strike line)

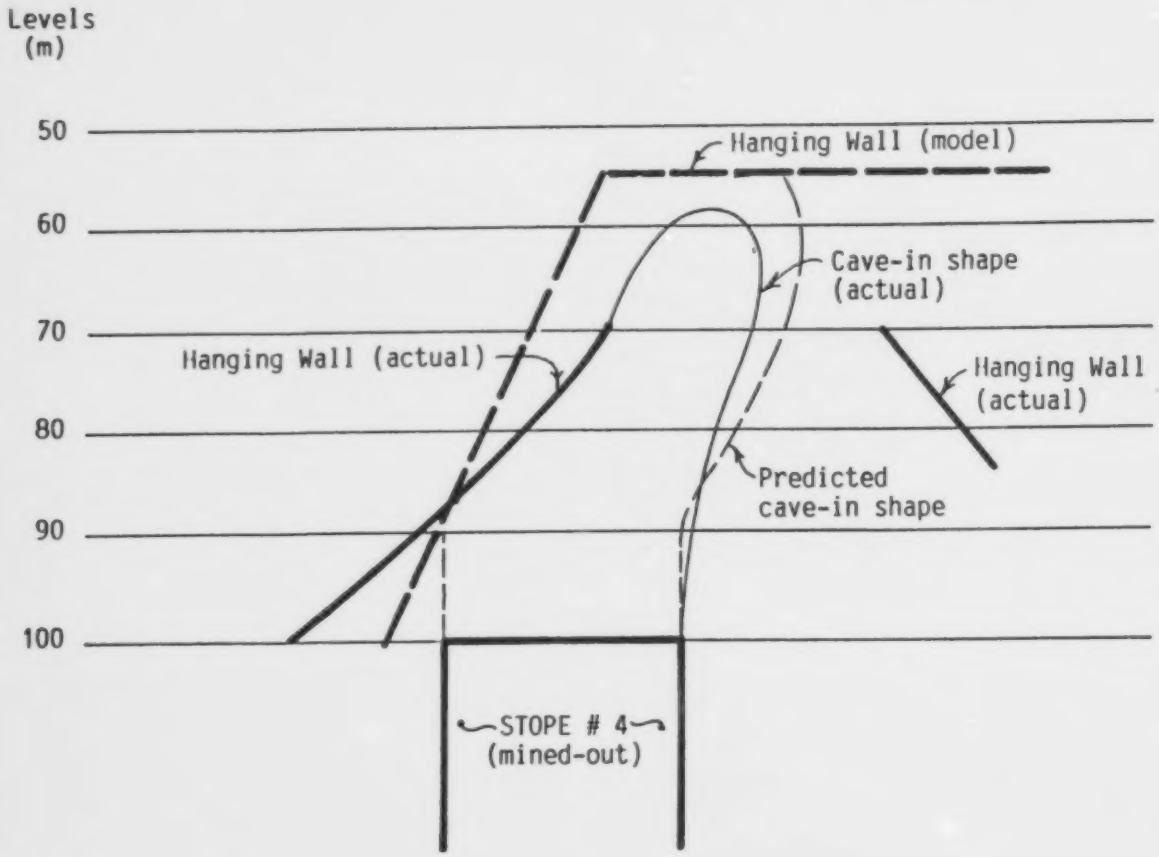


Figure 6-4 The Predicted and Actual Failure Surface- Cross Section B-B
(looking normal to Strike line)

In Fig. 6-3, the failure surfaces along cross-section A-A are shown. This figure indicates that the failure surface predicted by the analytical study is above the actual field failure surface. In Fig. 6-4, the field record and the predicted failure surfaces along cross-section B-B are plotted. Large differences in the two failure surfaces were observed. At the left hand side of the failure surface, the predicted failure surface is much higher than the field record. When the actual and simplified hanging wall are plotted on the same drawing, it is clear that in the field record, the failure did not go beyond the hanging wall. The predicted failure surface also stopped at the hanging wall.

The validity of the rock mass behaviour model is demonstrated by the favourable comparison of the field record and predicted failure surface of cave-in 'A'.

6.3.2 Time Dependence

It was understood that cave-in 'A' occurred about 6 to 8 months after the mining out of stope no. 4. This section of the Report discusses the likely cause of this delay in failure.

The kaolinized material found in the surface crown pillar is cohesive and has an activity of 1.25 which indicates that this material does not have a significant time dependent property.

One possible cause of the time delay failure at stope no. 4 is the variation of humidity and temperature in the stope.

A report by Bétournay(1980) on two decayed monzonite dikes from Mt. Royal, Montreal was examined. In that case, Monteregian related igneous activity in the form of dykes and sill are subjected to selective or complete decay. The weathering product is ultimately clay, kaolinite or illite depending on the extent of decay.

Exposures along highway cuts were examined. The weathering soils are interbedded in rock blocks. Failure occurs in a ravelling fashion which forms talus cones at the toe of the exposures. A wet enviroment is required for this type of decay process.

This case record from Mt. Royal indicates that the ravelling type failure may occur due to the weathering process which weakens the rock. In addition, during the winter season, water in the soil freezes and expands, and thaws in the spring. This cyclic freezing and thawing process loosens the soil. Such a frost mechanism may also be responsible for the observed failure in the Mt. Royal exposed slopes.

At Les Mines Selbaie, temperature and humidity changes alone can not explain the time delay failure. The cave-in occurred at the 100m level, where the rock in the mine has never been subjected to any freezing temperatures. The humidity in the stope may be subject to seasonal variations but such changes in humidity may only affect the surface few centimeters of the orebody. Enquiries were made of the Mine for information which may be available from their ventilation records. However, no substantive or quantitative information could be supplied by the Mine for the purpose of further investigation of temperature and humidity on the time delay observed in cave-in 'A'.

Other factors which may contribute to the time delay were investigated. One very important observation is that the soil is cohesive and difficult to mine by blasting. This indicates that immediately after blasting of the stope the material behaves in an undrained condition. Thus, all the induced tensile stresses due to mining out of the stope are taken up by the pore water in the form of negative porewater pressure. With time, the induced porewater pressures are dissipated and these stresses are transferred to the soil grains as effective stresses. In order to examine such a hypothesis, the time required for the excess porewater pressure to dissipate and the behaviour of the ground immediately after excavation need to be determined. This can subsequently be compared to the time taken for the caving process from start to finish.

Theortically, the elastic constants of cohesive soil under undrained condition can be estimated from those of drained elastic constants. In the undrained case, the Poisson's ratio is 0.5 and the Young's Modulus is approximately 20 per cent higher than the deformation modulus in a undrained condition (Lambe and Whitman, 1969).

With these undrained elastic constants, an analysis of the opening was conducted. No tensile failure zones were determined, implying that immediately after blasting, the stope is stable due to the set up of negative porewater pressure. With time, the porewater pressure dissipates, and failure occurs.

The time required for the porewater pressure to dissipate was calculated using a simplified two-dimensional analysis. This simplification is based on the assumption that the drainage path along the strike direction is much longer than the drain path along the vertical and across the strike directions. The drain path across the strike direction was assumed to be 20m. This is based on the assumption that water can be drained to the footwall fault, hanging wall fault and the orebody fault. The vertical drain path was taken to have a dimension of 50 m.

A typical coefficient of consolidation of 0.3 m2/day (Lambe and Whitman, 1979) was used for this study. The result of the analysis is shown in Table 6-2. The time required for 90 per cent consolidation is about 240 days. This time requirement compares favorably with the observed 6-8 month duration of cave-in "A".

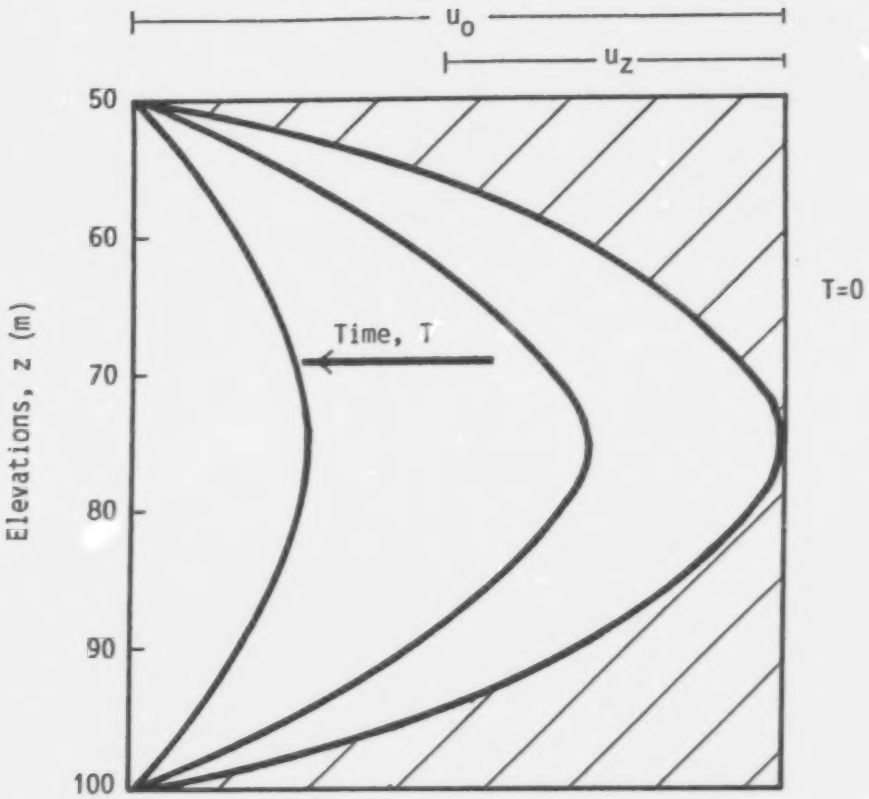
A schematic diagram of the vertical profile of porewater pressure distribution through the centre of stope no.4 is shown in Fig. 6-5. This diagram indicates that the porewater pressure changes rapidly in areas adjacent to the two drainage boundaries. This implies that cave-in will occur progressively upwards with time.

This proposed model only applies in cases where a time delay type of failure occurs. In areas where the kaolinized material is very soft, imminent type failures will occur immediately after blasting.

t (days)	U_H Degree of Consolidation - Horizontal Drainage (%)	U_V Degree of Consolidation - Vertical Drainage (%)	U_{HV} * Degree of Consolidation Due to Combined Drainage (%)
30	13	34	43
60	19	48	58
90	23	57	67
120	27	66	75
150	30	73	81
180	33	79	86
210	36	83	89
240	39	86	91

* $(1 - U_{HV}) = (1 - U_V)\ (1 - U_H)$

Table 6-2 Time vs. Degree of Consolidation



u_z - Pore pressure at elevation z

u_o - Initial pore pressure

U_i - Degree of Consolidation at elevation z

U - Average degree of consolidation

$$U_i = 1 - \frac{u_z}{u_o}$$

$$U = \frac{\text{shaded area}}{\text{total area}}$$

Figure 6-5 Pore Pressure Distribution as a Function of Mine Elevation

7.0 PARAMETRIC STUDY

Based on the weak rock mass behaviour model developed in the last chapter, the results of a parametric study are reported in this section. The objectives of this parametric study were to develop guidelines for the dimensioning of surface crown pillars at Les Mines Selbaie.

It had been decided by the Mine that the thickness of the surface crown pillar should be 7.5 m. The parameters considered in this section are therefore the thickness of the orebody and the width of the mining panels (or stopes) such that no cave-in will occur at the surface crown pillar after mining out of the panel (or stope).

From the existing field and laboratory data, the distribution of the kaolinized material can not be delineated. This problem is further complicated by the possible existance of isolated pockets of water in the overburden. The site has been dewatered by a drainage system; however, in areas where local pockets of water exist, the kaolinized material may be softened in time. In areas where such local zones occur at the surface crown pillar, the strength of the pillar will be greatly reduced. Thus, the width of the mining panel (or stope) may have to be reduced in order to avoid cave-ins or failures of the surface crown pillar.

In this parametric study, three types of material properties were considered. They are:

Ranking of Kaolinized Material	E (MPa)	v	Tensile Strength (MPa)
8	600	0.33	0.06
9	300	0.35	0.03
10	140	0.4	0.01

With these parameters, the width of the mining panels (or stope) to be used for safe mining were determined. The results of this study are plotted on Fig. 7-1. This figure clearly indicates that in areas where soft kaolinized material occurs, the width of the mining panel (or stope) will be very small. In such cases the width of the mining panel (or stope) could be increased with the use of rock bolting. Where bolting is used, Swellex type bolts are recommended, since this type of bolt provides good contact with the rock mass over the full length of the bolt.

Consideration will need to be given to the possible time effects of swellex bolts if the kaolinized zones consolidate. This consideration is vital to the selection of an appropriate rock bolting support system, but beyond the terms of reference of this study.

In areas where the orebody is thick, a wider mining panel (or stope) can be used provided the span of the mining panel (across the strike direction) is subdivided into sections. New sections can be mined-out when previouly mined-out sections have been completely backfilled.

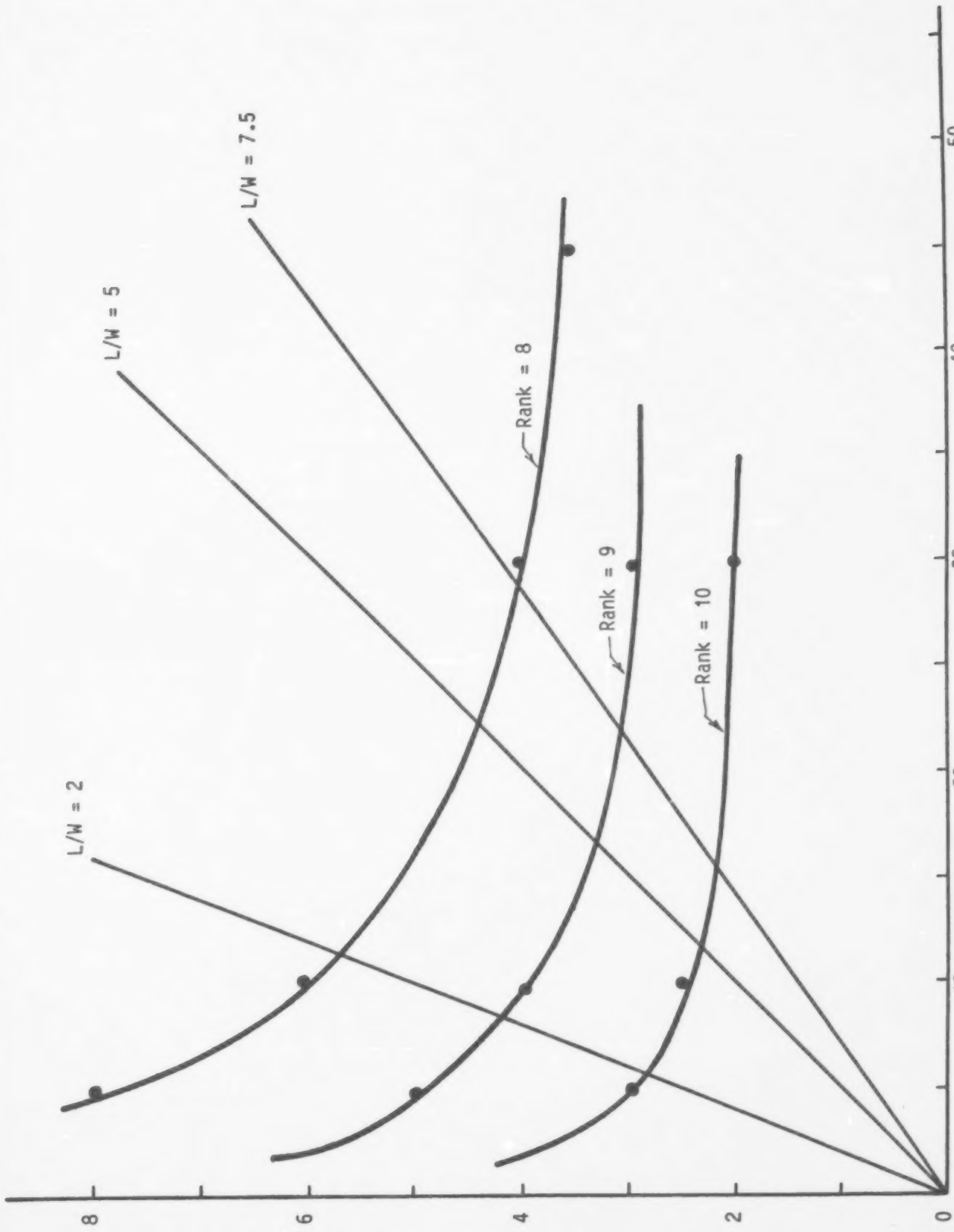


Figure 7-1 Design Guideline for Surface Crown Pillar at Les Mines Selbaie

Fig. 7-1 provides a guideline for the design of the width of surface crown pillars. It is very important to note that the use of this guideline should be supplemented with the experience of the mine engineers. With time, this set of curves may be modified through experience gained in the field.

Fig. 7-1 also indicates that with L/W ratios (the thickness of the orebody to the width of the mining panel) in excess of 7.5, a two dimensional analysis can be used for the dimensioning of the surface crown pillars.

8.0 CONCLUSIONS

In this Phase II study, a weak rock mass behaviour model has been developed. Based on this model, a computer program, ADINA, was used to analyse cave-in 'A' which occurred at the 100m level of stope no. 4. The predicted cave-in shape and the field records were compared. Good comparison was obtained. A rock mass failure mechanism was found.

With the validation of the weak rock mass behaviour model, a parametric study was conducted. Depending on the thickness of the orebody and the material properties of the surface crown pillar, a dimensioning guideline was established.

The conclusions of this study are as follows:

1. A 3-D non-linear behaviour model has been developed for the surface crown pillar kaolinized weak rock mass at Les Mines Selbaie.

2. Physical modelling was successful in modelling the cave-in shape from tensile failure and gravity fall for cave-in 'A' which occurred at the Mine.

3. The time delayed failure of cave-in 'A' can be explained by the induced negative pore water pressure in the kaolinized material caused by mining out of stope no. 4. With time, the negative pore water pressure dissipated and caving occurred progressively upwards. The duration of the actual cave-in was 6-8 months and this compares favorably with the time required for 90 percent consolidation to occur in the kaolinized material.

4. The results of the back analysis and the field record of cave-in 'A', indicate that the cave-in occurred progressively upward and become stabilized in the form of a compressive arch.

5. The strength of the kaolinized material is highly dependent on its water content. With the possible existence of local pockets of water in the overburden, local soft kaolinized zones exist.

6. Based on the ranking system developed in Phase I of this project, a design guideline on the dimensioning of the surface crown pillar at Les Mines Selbaie has been developed.

7. The design guideline developed in this study should be calibrated with field experience.

8. The results of the parametric study indicate that with a ratio of the thickness of the orebody to the width of mining panels (or stopes) greater than 7.5, a two-dimensional analysis can be used for the dimensioning of the surface crown pillar.

9. The width of mining panel (or stope) of the surface crown pillar increases with a decrease in the thickness of the orebody. This implis that a wider mining panel (or stope) could be used when the width of the panel (across the strike direction) is divided into sections. Each mined out section should be backfilled before mining of another new section.

10. The stability of the surface crown pillar can be enhanced with the use of rock bolts. Because of the variability of the material and the characteristic of the anticipated failure mode, Swellex type bolts may help achieve a continuous contact between the full length of the bolt and the rock mass. However, the potential for loss of contact with time due to consolidation of the kaolin zone must be taken into consideration.

11. A complete backfilling program should be implemented in the Mine. With such backfilling the width of the mining panel could be increased because of the development of negative pore water pressure (suction) at the surface crown pillar.

12. Because of the existance of a stratum of silty sand directly above the surface crown pillar, controlled caving methods may be considered for the future extraction of the surface crown pillar.

13. For most mines, surface crown pillars have been designed on the basis of empirical methods. It is proposed that the weak rock mass model developed in this study be used to study the data compiled by CANMET on other surface crown pillars. Through this proposed study, the developed model can be calibrated further to achieve a greater understanding and rationalization of the behaviour of surface crown pillars.

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10.0 CLOSURE

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A great deal of insight has been gained into the modelling and design of surface crown pillars in weak rock masses such as those at Les Mines Selbaie and we look forward to continuing the research into field monitoring of surface crown pillar during Phase IV.

Respectfully Submitted
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